B.1 TREATMENT BMPS

This Appendix provides design guidelines for the Caltrans approved Treatment Best Management Practices (BMPs) listed on Table 2-5 of this handbook. These BMPs have been approved for statewide use and should be considered for all projects that meet the criteria for incorporating Treatment BMPs, as described in Section 4 of this handbook.

B.1.1 Targeted Design Constituent

A Targeted Design Constituent (TDC) is a pollutant that has been identified during Departmental runoff characterization studies to be discharging with a load or concentration that commonly exceeds allowable standards and which is considered treatable by currently available Department-approved Treatment BMPs. The Targeted Design Constituent approach is the Department's statewide design guidance to address the "Primary Pollutants of Concern" as listed in the Figures 2-3 and 2-3(D7).

Targeted Design Constituents are: phosphorus; nitrogen; total copper; dissolved copper; total lead; dissolved lead; total zinc; dissolved zinc; sediments; general metals [unspecified metals]. A project must consider treatment to target a TDC when an affected water body within the project limits (or with the sub-watershed as defined by the Water Quality Planning Tool) is on the 303(d) list for the one or more of these constituents. Infiltration Devices, being the approved Treatment BMP capable of treating all the constituents listed on Table 2-2, Pollutants of Concern and Applicable Treatment BMPs, should be considered as the desired Treatment BMP for all watersheds in projects that are required to consider Treatment BMPs. However, if Infiltration Devices cannot be incorporated, or if the proposed Infiltration Device(s) cannot accept all of the WQV runoff that needs treatment, Biofiltration, Detention Devices, Multi-Chambered Treatment Train, Media Filter (Austin Sand Filter and Delaware Filter), and Wet Basins must be considered based on the Targeted Design Constituent approach. The remaining Caltrans-approved Treatment BMPs, Dry Weather Diversion, Gross Solids Removal Devices, and Traction Sand Traps, are applicable for specific situations as described in this Appendix and in this handbook.

B.1.2 Interaction with other Caltrans units

Besides Design, many other functional units may play a significant role in the implementation of the various Treatment BMPs into a project. These units should be consulted during the selection and design of Treatment BMPs. For example, District Landscape Architecture will select vegetative cover for many of the Treatment BMPs (e.g., Biofiltration BMPs), and should be consulted on siting issues for all the Treatment BMPs. District Maintenance must be consulted to insure that they can maintain the deployed BMPs. Proper hydraulic design is critical to the safe and efficient operation of all of the Treatment BMPs; this function is served by either the Project Engineer or by District Hydraulics depending upon the District and level of complexity of the design. Geotechnical Services will conduct site investigations for Infiltration and other Treatment BMPs. District Traffic Operations should be consulted when considering placement of Treatment BMPs in or near Clear Recovery Zones. The District Environmental unit plays a significant role in the environmental assessment of the project, and in the environmental clearance of sites for proposed Treatment BMPs. District NPDES and/or the Design Storm Water Coordinator plays a significant role by assisting in the interpretation of the PPDG, and by reviewing Storm Water Data Reports produced for the PID, PA/ED, and PS&E phases of the project. District Construction will help to identify potential constructability issues with the projects permanent Treatment BMPs, and will participate in the development of a Temporary (Construction) BMP strategy for the project. Other units may also have a role which is why it is so important to identify needed project information and to coordinate with those affected Functional Units early during each Project Phase.

B.1.3 Hydraulic Issues

Treatment BMPs are designed for water quality purposes, but they must also operate safely and effectively as part of the overall highway drainage system; because of this, hydraulic design issues must be carefully evaluated during the consideration and design processes for Treatment BMPs, especially with regard to any upstream effects that would impact highway drainage. While some aspects of hydraulic engineering are presented in this handbook, those presented will focus on the site-specific design of a Treatment BMP, and not on all aspects of hydraulic or hydrologic engineering. Instead, the Project Engineer is referred to the Highway Design Manual - Section 800, Highway Drainage Design, and he may require the assistance of the District Hydraulics Unit when situations arise for which it is advisable to route the flow through these Treatment BMPs to allow for better consideration of upstream and downstream effects (e.g., when a Treatment BMP is used for the dual purpose of peak flow attenuation and water quality treatment).

B.1.3.1 Design Events for Treatment BMPs

Several of the Treatment BMPs can be designed to work either on-line or off-line; for convenience, within the discussions of this text it is assumed that on-line placement will be made.[1] There are different potential impacts and design issues associated with on-line and off-line placement. If placed on-line, the general rule is that the design of a Treatment BMP uses either the Water Quality Volume or Water Quality Flow, but for compliance with the overall highway drainage system larger events must be considered as discussed in HDM Section 800. However, even if placed in an off-line configuration, some event greater than the WQ event must often still be considered for overflow or peak flow conditions. Under both placement conditions, freeboard should be maintained for Infiltration Basins, Detention Basins, and Wet Basins to prevent overtopping, as discussed in the respective subsections for each of these BMPs, and as discussed in HDM Section 800.

[1] When placed ‘on-line’, the BMP would be located in the drainage flow path of the runoff and the BMP must convey runoff from any storm that occurs by passing all flows through the BMP itself. Flows up to the WQV/WQF are treated by the BMP, while higher volume flows are safely passed through the basin without adversely impacting the upstream drainage systems, but without treatment. In contrast, ‘off-line’ Treatment BMPs systems primarily receive runoff from storm events up to and including the WQV/WQF, while larger events are mostly diverted around the Treatment BMP by an upstream device. Treatment BMPs which use WQV as the design basis must make an estimate of an equivalent flow rate to capture the 85th percentile runoff when designing the flow splitter for the off-line configuration.

B.2 BIOFILTRATION STRIPS AND SWALES

(VEGETATED TREATMENT SYSTEMS)

B.2.1 Description

Biofiltration strips are vegetated land areas, over which storm water flows as sheet flow. Biofiltration swales are vegetated channels, typically configured as trapezoidal or v-shaped channels, that receive and convey storm water flows while meeting water quality criteria and other flow criteria.

Pollutants are removed by filtration through the vegetation, sedimentation, adsorption to soil particles, and infiltration through the soil. Strips and swales are effective at trapping litter, Total Suspended Solids (soil particles), and particulate metals.

B.2.2 Appropriate Applications and Siting Criteria

Strips and swales should be considered wherever site conditions and climate allow vegetation to be established and where flow velocities will not cause scour. Vegetative cover of about 70% is required for treatment to occur. Biofiltration strips and swales should also be considered upstream of Treatment BMPs that would benefit from pretreatment by removing sediment loading, such as Infiltration Devices Detention Devices, and Wet Basins.

B.2.3 Factors Affecting Preliminary Design of Biofiltration Swales and Strips

B.2.3.1 Biofiltration Swales

Biofiltration Swales have two design goals: 1) to meet treatment criteria under Water Quality Flow (WQF) conditions, and 2) to provide adequate hydraulic function for flood routing and scour prevention for larger storm events by using Highway Design Manual Chapter 800-890 criteria. Treatment is maximized by designing the swale to be as gently sloped and as long as the site constraints allow.

For a swale to be designated as a Treatment BMP, criteria relating depth, velocity, and Hydraulic Residence Time (HRT) as presented in the formula below must be met:

$$\text{HRT}/(\text{depth} \times \text{velocity}) \geq C \qquad \text{(Eq. 1)}$$

where:

HRT = Hydraulic Residence Time during WQF, minutes (≥ 5 minutes)
depth = depth of flow at WQF (varies with velocity selected, up to 150 mm [0.5 ft])
velocity = velocity of flow at WQF (varies with velocity selected, up to 0.3 m/s [1 fps])
C = A constant: 0.22 for metric; 20 for US customary units

Note that the Hydraulic Residence Time is that time during which the WQF travels in the Biofiltration Swale, and has no relation to the Time of Concentration term as used in hydrologic calculations.

The Rational Formula should be used to calculate the runoff entering the bioswale as described in Topic 819.2 of the Highway Design Manual, using the appropriate Water Quality storm intensity from Section 2.4.2.2, Treatment BMP Use and Placement Considerations, of this handbook. Calculation of the depth of flow and velocity in the bioswale should be made using the Manning's equation, with the Manning's number under the WQF for preliminary calculations taken as n = 0.20 for "routinely mowed" strips and swales, at WQF Manning's n = 0.24 for "infrequently mowed" strips and swales. HEC 22, Tables 5-2 and 3 can also be consulted to determine an appropriate Manning's n for the site-specific depth if more rigorous calculations are deemed warranted.[2] In the situation where the WQF enters the proposed bioswale at a single upstream point, and only minor additional flow enters along the length of the swale, the calculation of Eqn. 1 is relatively simple. However, if the flow enters the Biofiltration Swale continuously along the length of the swale, or at multiple discrete locations, other rational methods should be employed. In the case of continuous flow entering the swale, the designer may wish to initially calculate the depths and velocities at selected points along the swale to verify that the depth or velocity has not exceeded the maximum allowed values. This same calculation could also be used if there is a change of grade. The length of the swale that would qualify as a Biofiltration Treatment BMP must be upstream of the location where either the maximum depth or velocity was exceeded. The calculation of the HRT when the WQF enters at multiple (actual entry points or discretized from continuous flow) entry points could be done by calculating the HRT for the flow from each of the discrete entry points, and then taking a weighted average of the HRTs for the entire flow over the length that qualifies as a Biofiltration Treatment BMP; velocity and depth criteria would still need to be met.

To provide adequate hydraulic function, a swale should also be sized as a conveyance system calculated according to criteria and procedures for flood routing and scour established in the Highway Design Manual Chapter 800.

Table B-1 summarizes preliminary design factors for biofiltration strips and swales.

B.2.3.2 Biofiltration Strips

Strips should be designed to be as long (in the direction of flow) and as flat as the site will allow to maximize treatment efficiency; while no HRT time has been assigned to Biofiltration Strips, and a 5 minute HRT should be sought if possible. The maximum strip length under which sheet flow conditions exist (and therefore treatment is obtained as a biostrip) is dependent on site conditions but may not exceed about 0.1 km [300 ft]. The area to be used for the strip should be free of gullies or rills that can concentrate overland flow and cause erosion.

Table B-1 summarizes preliminary design factors for biofiltration strips and swales.

[2] As a bioswale usually also conveys the HDM storm event (a much larger event than the WQF event), a more precise determination of Manning's n is usually unnecessary for water quality purposes.

Table B-1: Summary of Biofiltration Strips And Swales Siting and Design Factors

Description	Applications/Siting	Preliminary Design Factors
Swales are vegetated channels that receive and convey storm water as a concentrated flow. Strips are vegetated land areas over which storm water flows as sheet flow. Biofiltration treats the WQF. Treatment Mechanisms: • Filtration through the vegetation • Sedimentation • Adsorption to soil particles • Infiltration Pollutants primarily removed: • Litter • Total suspended solids • Particulate metals	• Site conditions and climate allow vegetation to be established – 70% minimum vegetation coverage will allow treatment, with better effects at higher coverage. • Consider locations for swales where flow velocities will not cause scour • Consider swales to provide pretreatment for other Treatment BMPs (infiltration devices, detention devices, and wet basins) • Eliminate from consideration sites with hazardous soils or contaminated groundwater plumes	• Strips and Swales: vegetation mix appropriate for climates and location • Strips and Swales: Use the Rational Method to determine the Water Quality Flow (WQF) and peak flows based on HDM Chapter 800 (often Q_{25}) • Swales designed as a conveyance system per HDM Chapters 800 to 890; • Swales: after designing to convey peak flows from HDM design storm, check swale against biofiltration criteria at WQF • Swales: design criteria under WQF: Hydraulic Residence Time of 5 minutes or more; maximum velocity of 0.3 m/s (0.9 ft/s); maximum depth of flow of 150 mm (0.5 ft), and Eqn. 1 relationship among these variables. • Swales: slope in direction of flow: minimum 0.25%, maximum 6%, with 1 to 2% preferred; • Swales: A minimum width (in the direction of flow) at the invert of a trapezoidal bioswale typically 0.6 m (2.0 ft); maximum bottom typically up to 3.0 m (10 ft); side slope ratio should be 1:4 or flatter; discuss bottom width and side slope ratio with District Maintenance. • Swales: consider if geosynthetic reinforcement of the bioswale would be helpful if flow velocity under the HDM event exceeds 1.2 m/s • Swales: freeboard: Confer HDM Topic 866 to determine if freeboard is required • Strips: sized as long (in direction of flow) and flat as the site will reasonably allow up to sheet flow boundaries (maximum length of Bio Strip is approximately 0.1 km [300 ft]); a HRT is not required, but a 5-minute HRT should be used if possible. • Strips: should be free of gullies or rills

B.2.4 Vegetative Factors

Apart from meeting the hydraulic parameters presented above, vegetation is the critical component in the effectiveness of Biofiltration Treatment BMPs. The District Landscape Architecture Office should be consulted for each project to recommend appropriate vegetation species. Every effort should be made to assure the successful establishment of vegetation, including consideration of the topics discussed below.

B.2.4.1 Soils

Soils with favorable infiltration characteristics promote successful vegetative cover by allowing healthy root development, thereby promoting the effectiveness of the biofiltration BMP. The Landscape Architect may recommend the following practices that foster infiltration and vegetation establishment:

- Stockpiling topsoil or duff prior to construction and replacement of topsoil in areas that will serve as biofiltration strips and swales;
- Cultivating and ripping of existing soils along the areas to be converted into biofiltration BMPS, to relieve compaction; and
- Incorporating soil amendments, including granular soils and organic material.

B.2.4.2 Selection of Plant Materials

Selection of plant materials for the biofiltration BMP should be based on the following:

- Tolerance to varying soil moisture, and an ability to survive during dry season without irrigation (unless irrigation is already in place or has been proposed with highway planting in adjacent areas);
- Long-term survivability that includes a mix of long-lived perennial species and annual species that successfully reseed;
- Dense, continuous root mass; and
- Dense, continuous top growth that includes grasses and grass-like species, forbs, and some broad-leafed species.

B.2.4.3 Plant Establishment

Seeded biofiltration strips and swales may require specific measures be incorporated in the design to ensure success. Consideration should be given to:

- Mulches, bark, straw, etc., on slopes are steeper than 1:4 (V:H) to improve infiltration and protect against surface erosion when no concentrated flow is present;
- Erosion control blankets to protect against surface erosion when concentrated flow is present;
- Turf Reinforcement Mat (TRM) or a suitable geosynthetic fabric as a bioswale lining for flows under the HDM Design Storm (design with assistance of Regional/District Hydraulics using methods listed in HDM Chapter 870);

- Temporary flow diversion to direct concentrated flow around newly seeded areas until vegetation is established;
- The use of sod may be preferred over seeding;
- If sod is used, supplemental water/temporary irrigation may be required during an establishment period; and
- An appropriate plant establishment period to ensure plant survivability.

B.2.4.4 Resources about Plant Materials

For additional information about native plant species suited to varied hydrologic conditions within specific ecological subregions of California, consult:

- Ecological Subregions of California Section and Subsection Descriptions, USDA, Forest Service, USDA, Natural Resources Conservation Service, published May 1998 (on line at: http://www.fs.fed.us/r5/projects/ecoregions/);
- Calflora Database (on line at: http://www.calflora.org); and
- Caltrans native grass database (on line at: http://www.dot.ca.gov/hq/LandArch/grass.html).

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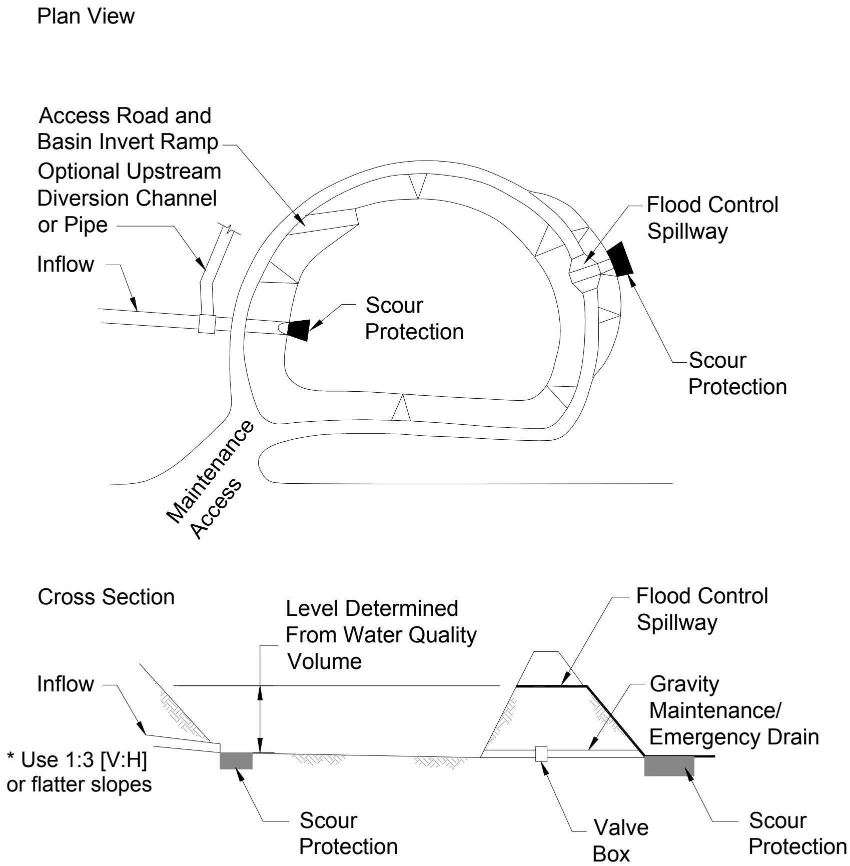
B.3 INFILTRATION DEVICES

An infiltration device is designed to remove pollutants from surface discharges by capturing the Water Quality Volume (WQV) and infiltrating it directly to the soil rather than discharging it to surface waters. Infiltration devices may be configured as basins or trenches.

B.3.1. Description

An infiltration basin temporarily stores the WQV while it infiltrates through the invert. An infiltration basin may be constructed in any shape to meet right of way restrictions. Runoff enters the basin under gravity flow. Storms greater than the WQV depth will overflow through a spillway if placed in an on-line configuration, but an infiltration basin must always incorporate an overflow spillway. A schematic illustration of an infiltration basin is shown in Figure B-1.

Figure B-1: Schematic of an Infiltration Basin
See Note 3



By contrast, an infiltration trench stores the WQV below ground prior to infiltration in the void spaces between rock placed in the trench. Infiltration trenches are often elongated, allowing

[3] Low flow channel not shown.

them to be used in constricted areas, but there is no shape restriction. A schematic illustration of an infiltration trench is shown in Figure B-2 (page B-11).

In order to avoid the classification of an Infiltration Trench as a regulated injection well, the infiltration trench should be designed as follows: a) the WQV should be directed to the infiltration trench by gravity flow in an open channel or as sheet flow; b) the captured volume should flow downward within the trench by the action of gravity, and without vertical piping for distribution to lower depths of the trench; and c) the widest dimension at the surface must exceed the depth of the trench.

Bypassing water from storms larger than the WQV upstream of the infiltration trench is preferred, as larger storms will usually generate sediment loadings larger than a WQV event, but larger events than the WQ design event must be allowed to pass downstream.

Performance of the infiltration trench is monitored using an observation well placed within the infiltration trench; this observation well can also be used to access the trench if drainage is required (using a hose and pump).

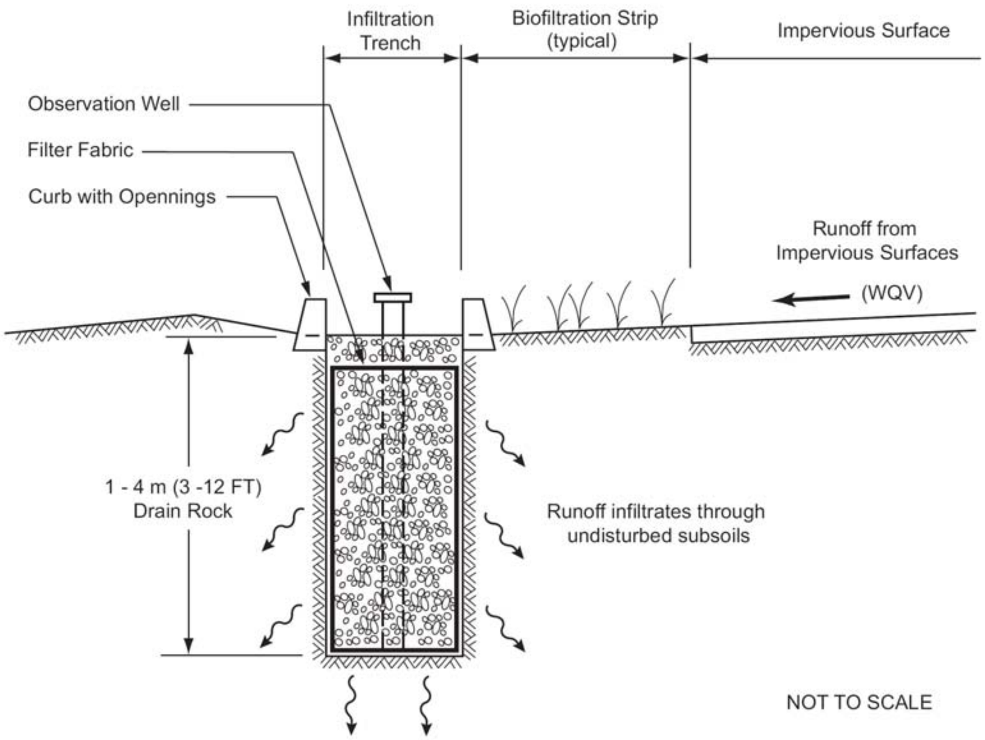
The required volume of the infiltration trench is quite large compared to the volume of an infiltration basin because the void space between the rock backfill holds the WQV, and that void space is typically only 1/3 of the total volume of the rock. Other high porosity backfill materials are available, thus reducing the volume of the trench; consult with the Headquarters Division of Environmental Analysis – Policy, Planning and Permitting, and Headquarters Design Office of Storm Water Management if such materials are under consideration for a site.

The typical configuration uses a filter-fabric lined trench (i.e., the trench is formed against bare earth with a fabric as a separator, rather than concrete walls) with a curb or dike at its perimeter at the ground surface; the filter fabric is employed between the rock and the native ground to prevent soil intrusion into the void space.

B.3.2 Appropriate Applications and Siting Criteria: Infiltration Basins and Trenches

Infiltration devices should be considered wherever site conditions allow and the design WQV exceeds 123 cubic meters (0.1 acre-foot). Appropriate sites for infiltration devices should have: a) sufficient soil permeability; b) a sufficiently low water table; c) the influent would not present a threat to local groundwater quality; and d) are at a sufficient elevation to allow gravity drainage of the device when needed for maintenance purposes. The Regional Water Quality Control Board (RWQCB) having jurisdiction may impose additional requirements for water protection purposes. Other physical siting conditions are discussed under Table 2, Applications/Siting. One other important siting requirement is that water stored in the infiltration basin, when constructed on-line, does not cause an objectionable backwater condition upstream in the storm drain system that would adversely impact its ability to convey design storms as required in the HDM.

Figure B-2: Schematic of an Infiltration Trench



Infiltration basins will function more effectively over the long-term if vegetated on the invert and side slopes. Consult the District Office of Landscape Architect for types of vegetation that can function effectively in Infiltration Basins in each of the various ecological subregions of a District. Additional information about grasses that have been successful within specific ecological subregions of California may be found in Ecological Subregions of California Section and Subsection Descriptions (as referenced in Appendix B, Biofiltration Strips and Swales).

Because an infiltration trench relies on flow through a filter fabric, the device is prone to clogging if fine sediments are allowed to enter the device. Rehabilitation of a clogged infiltration trench is difficult, especially compared to the relative ease to rehabilitate an infiltration basin. Because of this, pretreatment to capture sediment in the runoff is required upstream of the infiltration trench to increase longevity of the system (by using biofiltration devices or a forebay). To further minimize the clogging potential, the design may employ an upper layer of permeable material, typically about 150 mm in thickness, below which would be placed filter fabric, this upper layer would act as an initial filter, and could be periodically removed and replaced as conditions warrant (usually annually) rather than removing the entire rock volume.

Infiltration Trenches would likely be considered inappropriate for placement in High Risk areas, due to the difficulty in cleaning in the event of a spill; consult the District/Regional NPDES Coordinator if an infiltration trench is being considered in a High Risk Area.

B.3.3 Factors Affecting Preliminary Design

The following steps are recommended for determining the feasibility of infiltration device. The major components are Pre-screening, Site Screening, Site Investigation and Preliminary Design. Siting and design criteria are summarized in Table B-2.

Table B-2: Summary of Infiltration Device Siting and Design Criteria

(Applicable to both Infiltration Basins and Infiltration Trenches unless noted)

Applications/Siting	Preliminary Design Factors
• Infiltration Basin and Trench: Ability to treat a WQV ≥ 123 m³ (0.1 a-f); consult District/Regional NPDES Coordinator if an Infiltration Trench is being considered for a WQV between 80 and 123 m³. • Runoff quality must meet or exceed standards for infiltration to local groundwater • Infiltration devices should not be sited in locations over previously identified contaminated groundwater plumes • Separation from seasonally high water table > 3 m (10 ft), (or ≥ 1.2 m [4 ft] if justified by adequate groundwater observations for a minimum of 1 year); for most projects, the minimum clearance of 3 m should be provided; consult with District NPDES and Headquarters Design Office of Storm Water Management if < 3m clearance is being considered. • Soil types restricted to HSG A, B, or C (for Infiltration Basins) or HSG A or B (for Infiltration Trenches) having an infiltration rate ≥ 1.3 centimeters per hour (0.5 in/hr); maximum infiltration rate is 6.4 cm/hr (2.5 in/hr) unless a higher rate is approved in writing by RWQCB. For preliminary estimates of soil infiltration rate, consult Table B-3. • Soil should have a lay content < 30% and a combined silt/clay content < 40% • Site should not be located in area containing fractured rock within 3 m of invert [Table continues on next page]	• Infiltration Basins: Infiltrate WQV within 40 to 48 hours; Infiltration Trenches: Infiltrate WQV up to 72 hours • Use representative infiltration or permeability rate to size the device • Maintenance access (road around Basin and ramp to Basin invert, and to the Trench) • Infiltration devices should not be placed in service within a construction contract until all upstream runoff is stabilized, or shall be protected from sediment-laden runoff. • Infiltration Basins: Optional upstream diversion channel or pipe for storm events > WQV; mandatory downstream overflow structure as part of the Basin flow control device sized to pass the largest design storm event (up to the 100-yr storm) that will enter the basin, minimum spillway length 1 m (3.3 ft), as overflow weir or outlet riser • Infiltration Basins: Provide a minimum 300 mm (12 in) freeboard (the elevation between the top of the confinement forming of the Basin and the elevation of the water under the largest storm that can enter the basin • Infiltration Basin: Scour protection on inflow and spillway • Infiltration Basins: Use as flat an invert as possible (3% maximum); Infiltration Trenches: flat invert (no slope) • Infiltration Basins: Provide emergency/maintenance gravity drain, if practicable • Infiltration Basins: Use 1:4 side slope ratios or flatter for interior side slopes, unless approved by District Maintenance, with 1:3 maximum • Infiltration Basins: Provide vegetation, typically grasses at invert and side slopes • Infiltration Basin: Provide an emergency/maintenance gravity drain, 200 mm diameter (8 inches) [Table continues on next page]

Table B-2: Summary of Infiltration Device Siting and Design Criteria (cont.)

(Applicable to both Infiltration Basins and Infiltration Trenches unless noted)

Applications/Siting	Preliminary Design Factors
• Locate where sloping ground < 15%, and where infiltrated water is unlikely to affect the stability downgradient of structures, slopes, or embankments • Locate at least 300 m (1,000 ft) from any municipal water supply well; at least 30 m (100 ft) from any private well, septic tank or drain field; and at least 60 m (200 ft) from a Holocene fault zone • Locate > 3 m (10 ft) downgradient and 30 m (100 ft) upgradient from structural foundations, when infiltrating to near surface groundwater. • Wetting front water level should not cause groundwater to rise within 0.2 m (0.7 ft) of the roadway subgrade; • Infiltration Trenches: installed down-gradient from the highway structural section, and should not be placed closer horizontally than the Trench depth to the roadway if in a location subject to frost • Infiltration Trenches: would likely be considered inappropriate for placement in High Risk areas, due to the difficulty in cleaning in the event of a spill; consult District/Regional NPDES Coordinator if an infiltration trench is being considered in a High Risk Area. • Locate outside the 9 m (30 ft) Clear Recovery Zone, or consult with Traffic Operations to determine if guard railing is required	• Infiltration Trenches: total volume ≥ 3x WQV • Infiltration Trenches: Provide one observation well in the Trench, minimum diameter of 150 mm, with weatherproof cap; may be used to drain the trench if necessary. • Infiltration Trenches: maximum depth of trench is 4 m, depth less than the widest surface dimension, and WQV should be directed to trench as surface flow, and allowed to gravity-flow downward to the invert of the trench. • Infiltration Trench: use rock specified elsewhere in this section; a 150 mm (6 inches) layer of Permeable Material (Standard Specification 68-1.025) is usually placed at the invert to protect the filter fabric from the rock during its placement. • Pretreatment to capture sediment in the runoff (such as with biofiltration or a forebay): required for Infiltration Trenches, and recommended for Infiltration Basins. Only approved BMPs should be considered. • Infiltration Trenches often have a perimeter curb for delineation, and to limit vehicle wheel loads from encroaching upon the trench; may use A1-150 (Standard Plan sheet A87).

Rock meeting Rock Slope Protection, Method B Placement, Class 3 (Standard Specification 72-2.02, "Materials") should be used in Infiltration Trenches with the following gradation.[4]

Sieve Size, mm	Per cent passing
100	100
75	50 - 100
50	20 - 85
38	10 - 75
25	5 - 40

B.3.4 Pre-Screening for the Infiltration Device

Pre-screening for the infiltration device involves collecting site-specific information necessary to determine whether infiltration is an appropriate storm water treatment method and to ensure the site meets criteria established by the RWQCB. Consult with the District/Regional NPDES Coordinator to obtain RWQCB criteria. No field testing is anticipated during this early investigation.

4 Minor variation from these gradations will have little effect on the void space available.

The steps involved in pre-screening include:

- Information collection; and
- Preliminary determination of infiltration appropriateness.

The following sub-sections describe the steps involved.

B.3.4.1 Information Collection

Some of the basic site-specific data required for the determination of the appropriateness of the infiltration BMP are found in the sources listed below. Additional data may be required for local conditions. Data collected by Caltrans project engineering staff and Caltrans District/Regional NPDES Storm Water Coordinators include, but may not be limited to:

- Outfall inventory data available through District/Regional NPDES Storm Water Coordinators, project alignment, right-of-way, annual average daily traffic (ADT), Caltrans outfall locations, and other basic project maps and data;
- Tributary drainage areas and surrounding land uses (from outfall inventory, as-builts, aerial photographs, Geographic Information System (GIS) data from Caltrans and local planning agencies, etc.);
- Site surface hydrology data: tributary drainage area, runoff coefficients, drainage network, travel times, etc., needed to design facilities to Caltrans hydrologic/hydraulic criteria;
- Basin Plan groundwater beneficial uses and known impairments (RWQCB).
- Caltrans runoff quality data for appropriate Caltrans land use in catchment area (Caltrans Annual Report or Caltrans Monitoring Sites http://www.stormwater.water-programs.com/Research.htm stormwater.water-programs.co); and
- WQV calculated in accordance with Section 2; the program Basin Sizer satisfies the requirements of Section 2 and is available at http://www.stormwater.water-programs.com/Research.htm

Site soil characteristics:

- Indigenous soil types: Natural Resources Conservation Services (NRCS) soil maps and corresponding hydrologic soil classes, USCS classifications, or similar;
- Soil infiltration rates (estimated and from any existing on-site testing in the vicinity by others); and
- Caltrans project grading plans or as-built plans (if retrofit), if available.

Existing groundwater and hydrogeology information:

- Maps of local aquifers underlying the alignment or location of the proposed Caltrans project; and

- Aquifer groundwater quality and seasonal groundwater levels: monitoring well data, U.S. Geological Survey (USGS), Department of Water Resources (DWR), and local public agency maps and databases (e.g., http://wdl.water.ca.gov/gw/admin/main_menu_gw.asp)

Local groundwater quality concerns: Consult RWQCB, California Department of Toxic Substances Control (DTSC), local environmental/health department (city/county);

- Site hydrogeology (from any existing boring logs: lenses, hardpan, etc.);
- Known contaminated groundwater plumes (RWQCB);
- Groundwater rights data: adjudicated groundwater basins, other rights (RWQCB, DHS); and
- State Water Information Management System data for project area (State Water Resources Control Board [SWRCB]).

B.3.4.2 Preliminary Determination for Appropriateness of Infiltration

Once the data have been collected and placed in the context of the alignment and/or location of the Caltrans facility being considered for infiltration devices, the Project Engineer and the District/Regional NPDES Storm Water Coordinator will use the data and follow the procedure outlined in Figure B-3 (page B-17). Infiltration Devices being considered for District 7 should also apply the procedures outlined in Figure B-20 (page B-69).

Applicable steps for determination of appropriateness of infiltration include:

1) Determine if local Basin Plan or other local ordinances provide limits on quality of water that can be infiltrated. Compare with Caltrans runoff quality, and determine if infiltration is permissible. If not, document inapplicability of infiltration and continue to step 5 for consideration of other approved Treatment BMPs.
2) Determine if local agencies, public health authorities, legal restrictions, or other concerns preclude consideration of infiltration of storm water runoff. Consult with District/Regional NPDES Storm Water Coordinator and representatives of appropriate authorities as needed. If infiltration into the aquifer is not acceptable to local authorities, document inapplicability of infiltration, and continue to step 5 for consideration of other approved Treatment BMPs.
3) Estimate the quality of runoff from the Caltrans facility draining into the proposed infiltration device using data from the Caltrans storm water database and annual research summaries.
4) Compare the estimated Caltrans runoff water quality with available groundwater quality data, using receiving water objectives from the RWQCB Basin Plan, for each groundwater beneficial use. Determine if the separation between the maximum anticipated seasonal high groundwater table and the proposed basin invert is at least

3 m (10 ft). Tabulate the results and make a preliminary determination of the appropriateness of the infiltration BMP.

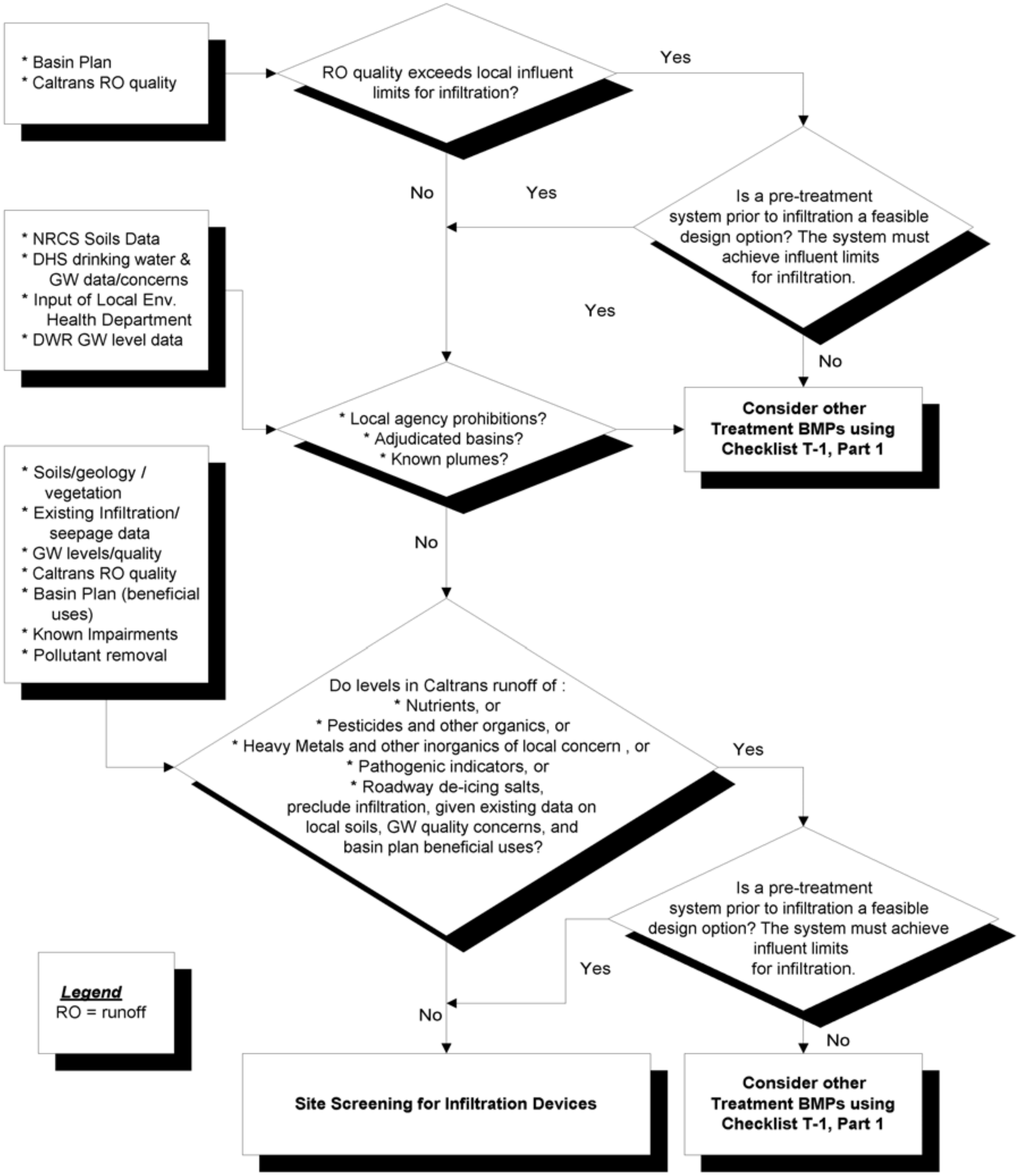
5) If the determination is negative (i.e., infiltration *not* appropriate), consider other approved treatment BMPs according to the Targeted Design Constituents (TDC) approach as defined in the Storm Water Data Report (SWDR). If determination is positive (i.e., infiltration potentially appropriate), proceed to infiltration site screening.

B.3.5 Infiltration Device Site Screening

Using data gathered in the pre-screening process, perform an initial screening of sites to narrow the number of potential sites to those that can be considered for field investigations within the project limits. As needed, collect additional information, and follow these procedures:

- Estimate soil type (consider NRCS Hydrologic Soil Groups [HSG] A, B, or C only for Infiltration Basin, HSG A or B only for Infiltration Trench, as shown in Table B-3) from soil maps and/or U.S. Department of Agriculture (USDA) soil survey tables and/or background information. In areas where septic systems are in widespread use, the County Environmental Health Department may have information on appropriate soil types for infiltration of on-site wastewaters;
- Also review other key available data: percent silt and clay, presence of a restrictive layer, permeable layers interbedded with impermeable layers, and seasonal high water table. Other geotechnical considerations that may restrict usage include: location in seismic impact zones, unstable areas, such as landslides and Karst terrains, and those with soil liquefaction and differential settlement potential, or highly expansive/collapsible soils. Generally, sites should not be constructed in fill, or on any slope greater than 15 percent;
- Also review other key available data: percent silt and clay, presence of a restrictive layer, permeable layers interbedded with impermeable layers, and seasonal high water table. Other geotechnical considerations that may restrict usage include: location in seismic impact zones, unstable areas, such as landslides and Karst terrains, and those with soil liquefaction and differential settlement potential, or highly expansive/collapsible soils. Generally, sites should not be constructed in fill, or on any slope greater than 15 percent; and
- The minimum acceptable spacing between the proposed infiltration device invert and the maximum seasonal high groundwater table is 3 m (10 ft). If a separation of less than 3 m is proposed, the approval of the local RWQCB is required.

Figure B-3: Pre-screening for the Infiltration Devices



Consultation to take place between RWQCB and Caltrans staff to evaluate RO quality data against basin plan and other local/regional GW concerns to reach site-specific "go-no go" decision on Infiltration Device BMP.

Table B-3: Typical Infiltration Rates for NRCS Type, HSG, and USCS Classifications

NRCS Soil Type	HSG Classification	USCS Classifications See Note 1/	Typical Infiltration Rates See Note 2/	
			cm/hr	(in/hr)
Sand	A	SP, SW, or SM	20	(8.0)
Loamy sand	A	SM, ML	5.1	(2.0)
Sandy loam	A	SM, SC	2.5	(1.0)
Loam	B	ML, CL	0.8	(0.3)
Silt loam and silt	B	ML, CL	0.6	(0.25)
Sandy clay loam	C	CL, CH, ML, MH	0.4	(0.15)
Clay loam, silty clay loam, sandy clay, and silty clay	D	CL, CH, ML, MH	<0.2	(<0.05)
Clay	D	CL, CH, MH	<0.1	(<0.05)

Note 1: USCS classifications are shown as approximation to the NRCS classifications. Note that the NRCS textural classification does not include gravel, while the USCS does. Note also that the gradation criteria (particle diameter) for the three soil types as used in the NRCS and the USCS, while agreeing in large part, are not congruent. Dual classifications in the USCS omitted. Infiltration estimates for USCS found in standard geotechnical references may vary from those shown for NRCS classifications, especially if significant gravel is present.

Note 2: Infiltration basins should be placed at locations with soils classified as HSG A or B, although C soils can be acceptable if geotechnical investigations demonstrate minimum infiltration rate of 1.3 cm/hr (0.5 in/hr). Infiltration trenches should be placed at locations with soils classified as HSG A or B and that have a minimum infiltration rate of 1.3 cm/hr. Maximum infiltration rate allowed for any infiltration device is 6.4 cm/hr (2.5 in/hr) unless RWQCB approval is received.

Infiltration devices should not be sited in locations over previously identified contaminated groundwater plumes; setback distance should be determined in coordination with the RWQCB.

Estimate infiltration rate for the soil type at the site using Table B-3.

Estimate the area required for an infiltration basin as follows:

$$A_{est} = (C \times SF \times WQV)/(k_{est} \times t) \qquad \text{(Eq. 2)}$$

where:

A_{est} = estimated area of invert of basin (m^2 or ft^2)
C = conversion factor (100 for cm to m; 12 for inches to ft)
SF = safety factor of 2.0
WQV = Water Quality Volume calculated from the design storm (m3 or ft^3)
k_{est} = estimated or representative infiltration rate from Table B-3 (metric: cm/hr; US customary units: inches/hr)
t = drawdown time, 40 to 48 hours

Estimate the invert area required for an infiltration trench as follows:

$$A_{est} = [C \times SF \times WQV]/[(0.35k_{est} \times t] \qquad \text{(Eq. 3)}$$

where:

Aest = estimated area of invert of basin (m^2 or ft^2)
C = conversion factor (metric: 100 to convert from centimeters to meters; customary US units: 12 to convert from inches to feet)
SF = safety factor of 2.0
WQV = water quality volume calculated from the WQ design storm (m^3 or ft^3)
kest = estimated or representative infiltration rate from Table B-3 (metric: cm/hr; customary US units: inches/hr)[5];
t = drawdown time, up to 72 hours
0.35 = porosity of void material (value for rock shown)

Once the area is obtained, the length (L) and width (W) can be calculated for a given D:

$$D = WQV/0.35A \qquad \text{(Eq. 4)}$$

where

D = trial depth of the trench (≤ 4 m [13 ft])[6]
WQV = water quality volume calculated from the WQ design storm (m^3 or ft^3)
A = estimated or calculated area of invert of infiltration trench (m^2 or ft^2)
0.35 = porosity of void material (value for rock shown)

Then, to calculate the length and width:

$$WQV = 0.35L \times W \times D \qquad \text{(Eq. 5)}$$

with L and W sized to meet site constraints.

B.3.6 Site Investigation

After the desktop screening of sites has been completed (including those sites outside of existing Caltrans right of way), proceed with field investigations of the remaining potential sites.

- Perform site investigation to identify any: (a) Regulatory permit required, (b) Major underground utility interference, (c) Transportation improvement plan conflicts, or (d) General plan land use data for tributary area;
- If considering a parcel outside of the right-of-way, Caltrans must generate greater than 50% of the total tributary runoff directed toward that parcel; otherwise

[5] Note that native soil around the infiltration trench must be from HSG Group A or B

[6] The typical depth of an infiltration trench is between 1 and 3 meters, with a maximum of 4 m. To avoid classification as an underground injection well, the infiltration trench at its widest dimension (length or width) must exceed its depth.

investigate opportunities for a cooperative agreement to share storm water treatment facilities with the other agency, county, or city responsible for the additional flow;

- Assess the feasibility (e.g., degree of plumbing required and available area) of directing runoff from additional tributary area to the device; additional Caltrans area would have priority; other off-site areas are secondary. Consider potential downstream impacts from diversions and cost of diverting additional flow. Diversions of runoff from outside the tributary area of the infiltration device to unimproved conveyances (creeks/streams) are prohibited due to the increased potential for erosion. Diversions to improved conveyances may be permitted if it can be demonstrated that the conveyance has sufficient capacity to accommodate the additional flow, and other environmental considerations are favorable or neutral. If such diversion is being considered, consult with District/Regional Environmental and Hydraulics units;
- Investigate feasibility of infiltration using criteria and the procedure in Section B.3.4.1. Recalculate and verify area requirements using the collected field data. Use Equation 2 (see Section B.3.5) and the lowest measured or anticipated infiltration rate, or value considered representative of by the geotechnical professional, to calculate area of the infiltration device; and
- If an infiltration device is feasible, proceed to Section B.3.7, Preliminary Design.

B.3.6.1 Procedure for Preliminary Infiltration Device Site Investigation

The following scope of work defines the steps for infiltration device studies necessary to determine if an infiltration device may be feasible on the subject site. The screening procedure is terminated if the site does not meet the criteria for any step, and assessment of the site would continue, but for other approved Treatment BMPs. Geotechnical site investigations may be difficult to schedule, and might be conducted during the Design phase.

The depth to groundwater must be known as a first step in feasibility because a high groundwater table can lead to infiltration failure and potential contamination of the groundwater table. The *in situ* infiltration rate at the device invert must also be known or reasonably estimated to ensure that infiltration of the calculated WQV is possible within 48 hours. Due to the extreme variability of site conditions, field investigation is almost always required to determine the depth to groundwater and to provide an evaluation of the *in situ* infiltration rate.

The scope of work comprises two phases:

- Initial Investigation; and
- Detailed Investigation as follows.

Initial Investigation

The initial investigation comprises two parts: A) Initial technical field screening and determination of groundwater elevations, and B) Geotechnical investigation for soil lithology and select chemical testing. To streamline the initial investigation phase, Part A will be performed first, followed by Part B if the Part A criterion of at least 3 m (10 ft) clearance for the groundwater elevation below the device invert is satisfied and the site is deemed appropriate for further consideration. Consult the local RWQCB for approval of proposed groundwater separation less than 3 m (10 ft).

Part A: Initial Technical Field Screening and Determination of Groundwater Elevation

A local or regional groundwater review will be performed based on the available data, including, but not necessarily limited to:

- Previously compiled databases on potential BMP sites (such as outfall inventory databases);
- Data and maps available from regional government databases, DWR, other local agencies and internal Caltrans sources;
- Local soil survey data from the NRCS and other sources;
- Soil lithology, infiltration rate and groundwater depth data from the county or other specialists that approve septic system installations in the local area;
- Information on local groundwater beneficial uses and groundwater quality issues from the RWQCBs and other water resource agencies; and
- Information on local groundwater-related drinking water issues from DHS.

An initial indication of the seasonal high groundwater water table elevation will be determined by using a piezometer, previous studies, or other accepted geotechnical means. The piezometer will be installed to a depth of at least 6 m (20 ft) below the proposed device invert using the direct push or other suitable method. Initial groundwater levels will be recorded at least 24 hours after installation.

The geotechnical professional will make a determination on a site-by-site basis, whether the groundwater elevation determined after 24 hours can be considered to be a reasonable indication of the seasonal high water table for the purposes of the evaluation of the groundwater depth criteria, described as follows. If such determination cannot be made reasonably based on the available data, the site will be recommended for a longer period of water table elevation monitoring, as necessary.

If the initial seasonal high groundwater elevation indication is within 3 m (10 ft) of the invert of the proposed infiltration device, <u>the site will be eliminated from further consideration unless the local RWQCB requires installation of an infiltration device with less than 3 m separation to groundwater.</u> If there is not a reliable indication that the seasonal high water table is at least 3 m below the invert of the proposed infiltration device (i.e., if there is reason to believe the water table may rise to within 3 m of the proposed invert), a more extensive groundwater table elevation investigation will be performed as described in Section B.3.4.2, Part C. If the groundwater elevation at the site is clearly deeper than 3 m from the proposed device invert and all other criteria in the initial investigation are satisfied, a detailed groundwater elevation determination will not be required.

Part B. Geotechnical Investigation for Soil Lithology and Select Chemical Testing

An initial soil investigation will be performed to adequately evaluate soil lithology and determine:

- If there are potential problems in the soil structure that would inhibit the rate or quantity of infiltration desired; or
- If there are potential adverse impacts to structures, slopes or groundwater that could result from locating the infiltration device at the site to structures, slopes or groundwater.

Geotechnical trenches (a boring may be used at the option of the geotechnical professional) will be dug using a backhoe at one or two locations within each site, depending on the site conditions. Clearance of the site for hazardous contaminants through the appropriate District should be done prior to drilling by the geotechnical professional conducting the work; Underground Service Alert (USA) clearance will also be obtained. The trenches will be at least 2 m (6 ft) long and 2 m (6 ft) deep below the proposed device invert. The soil profiles will be carefully logged to determine variations in the subsurface profile. Of greatest importance is the presence of fine-grained materials such as silts and clays, which should be determined by direct measurement of particle size distribution. Two to four soil samples should be collected for determination of the soil particle size distribution at each site. Samples should be collected from the soil profiles at different horizons and transported to a laboratory for soil indices testing, plasticity, and chemical testing described as follows:

- Soil textures or classifications that are conducive to infiltration include sands, loamy sands, sandy loams, loams, silt loams, and silt in the NRCS classification system, or GW, GM, SP, SW GC, SC, SM, and ML (in the Unified Soil Classification System) as long as the soil does not have more than 30 percent clay or more than 40 percent of clay and silt combined; and
- The soil in the first 300 mm (12 inches) below the basin invert will be tested for organic content (OC), pH, and cation exchange capacity (CEC) only if required by the local approving agency (notify Geotechnical Services prior to site investigation for this testing). Values that promote pollutant capture in the soil are: OC > 5%, pH in the range of 6-8, and CEC > 5 meq/100 g of soil (however, soils that have this CEC value are typically fine-grained, and often would be rejected for infiltration based on permeability considerations).

In addition, the trenches or samples from borings, should be examined for other characteristics that may adversely affect infiltration. These include evidence of significant mottling (indicative of high groundwater), restrictive layer(s), and significant variation in soil types horizontally and vertically. A summary report will be prepared addressing the issues noted in this section, with recommendations on the suitability of the site for infiltration and the necessity of carrying out the next phase of the investigation. (All the site reports will ultimately be combined in a single report.) The geotechnical professional will develop the detailed investigation phase for the sites deemed acceptable from the initial investigation.

B.3.6.2 Detailed Investigation

If the site conditions still appear favorable to infiltration after the geotechnical review and soil investigations, a detailed field investigation will be undertaken, which includes Part A, Detailed Subsurface Soil Investigation, Part B, Permeability Testing, and Part C, Detailed Groundwater Elevation Determination.

Part A. Detailed Subsurface Soil Investigation

Borings will be drilled to a maximum depth of 15 m (50 ft) (or refusal in rock or rock-like material at a lesser depth), and to a minimum depth of 3 times the depth of water when in the basin (at the WQV depth) for each detailed investigation location. Samples will be obtained at 1.5-m (5-ft) intervals for soil characterization and/or laboratory testing. Bulk samples will also be collected at shallow depths (i.e., just below the invert elevation) to verify information collected in Parts A and B of the Initial Investigation.

Part B. Permeability Testing

No single test method is appropriate for the variety of subsurface conditions that might be encountered, as, for example, a percolation test at the invert elevation might not disclose the existence of layers of either highly permeable or low permeability within the depth of interest. Rather, a permeability evaluation below the invert of the proposed infiltration device will be made using infiltration rate tests or other method(s) selected by the Geotechnical Professional.

The minimum acceptable infiltration rate for an infiltration device is 1.3 cm/hr (0.5 in/hr). If any test hole shows less than the minimum value, the site will be disqualified from further consideration unless strong local geotechnical evidence exists to predict the successful performance of the device. If the infiltration rate at the site is greater than 6.4 cm/hr (2.5 in/hr), the RWQCB must be consulted, and the RWQCB must conclude that the groundwater quality will not be compromised, before approving the site for infiltration.

If the site is constructed in fill or partially in fill, it will be excluded from consideration unless no silts or clays are present in the soil boring within 4m (13 ft) of the device invert; fill tends to be compacted, with clays in a dispersed, rather than flocculated state, greatly reducing permeability.

The geotechnical investigation will be sufficient to develop an adequate understanding of how the storm water runoff will move in the soil (horizontally or vertically), and if there are any geological conditions that could inhibit the movement of water.

Part C. Detailed Groundwater Elevation Determination

If a detailed investigation to determine the groundwater elevation is required per the guidance and, in the opinion of the geotechnical professional, the seasonal high groundwater elevation may come within 3 m (10 ft) of proposed device invert, at least one groundwater monitoring well will be installed at a representative location. The well(s) will be observed over a wet and dry season. This observation period will be extended to a second wet season (at the direction of Caltrans) if the first wet season produces regional rainfall less than 80% of the historical average. The minimum acceptable spacing between the proposed infiltration device invert and the seasonal high water table is 3 m (10 ft), unless, in coordination with the RWQCB, it can be demonstrated that the groundwater will not be adversely impacted. A geotechnical professional will oversee the detailed investigation and must also consider other potential factors that may influence the groundwater elevation, such as local or regional groundwater recharge projects, future urbanization, or agricultural practices. The geotechnical professional should also examine the soil borings for indications of previous high water.

A final geotechnical report, overseen by a geotechnical professional, summarizing the findings of the investigation will be prepared. The report will include all results from the initial as well as detailed investigation phases of the feasibility study.

B.3.7 Preliminary Design

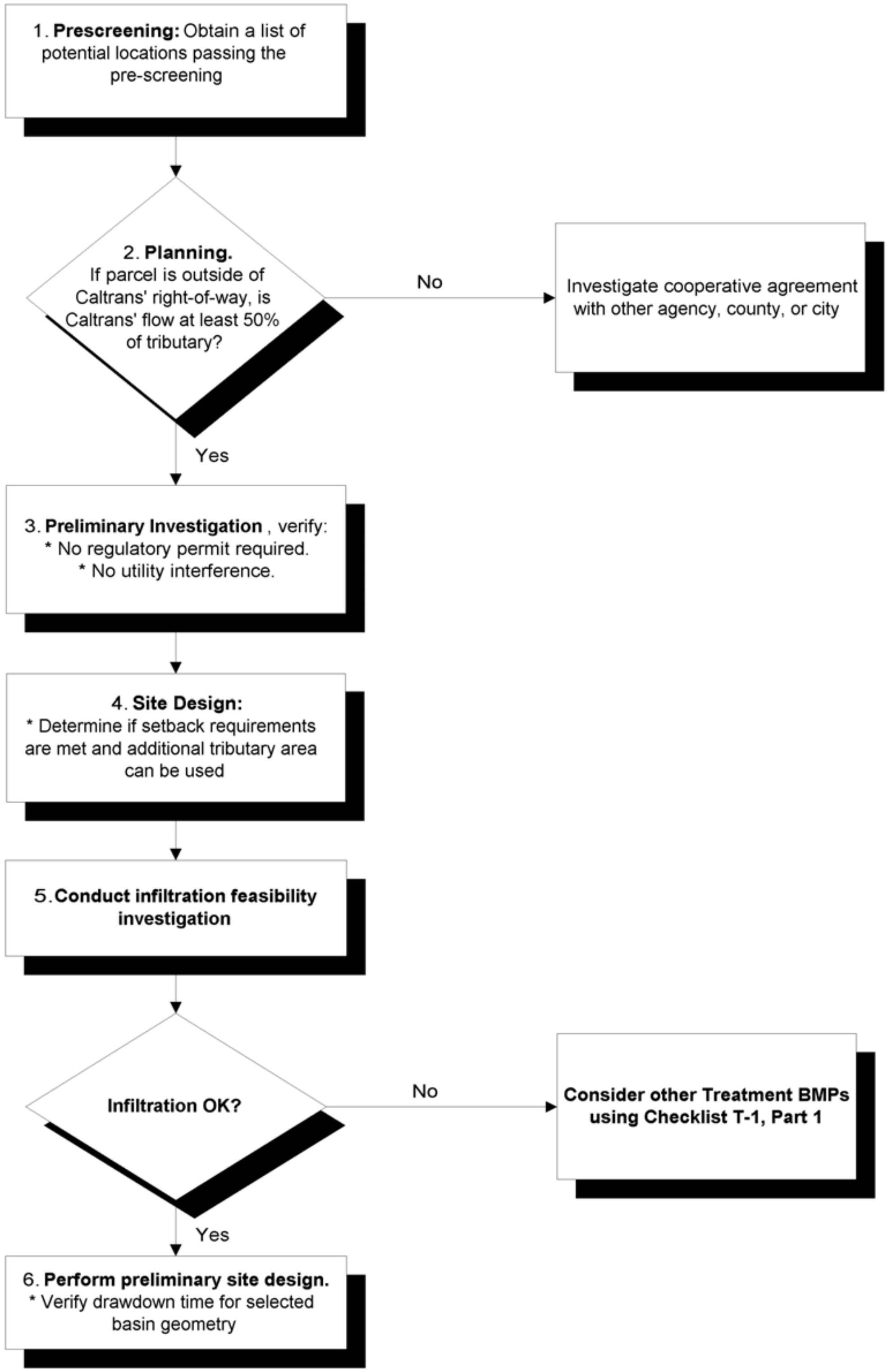
Table B-2 summarizes preliminary design factors for infiltration devices. Preliminary design includes the following:

- Obtain site topography (one-half meter contours, 1:500 scale). Extend topography 25 m (80 ft) beyond the infiltration device perimeter to show where runoff enters or leaves Caltrans right-of-way, enters a drainage channel owned by others, or enters a receiving water;
- Develop a conceptual grading plan for improvements showing the device, maintenance access, device outlet and extent of right-of-way requirements to accommodate the improvements. The device invert must not have a slope of greater than 3%;
- Develop unit cost-based cost estimate to construct the infiltration device. Include allowances for traffic management and storm drain system improvements as needed and determined by the PE; and
- Develop single paragraph assessments of: nonstandard design features; impact on utilities; hydrology (WQV, peak flow, land use); right-of-way total area needed; current ownership; highway planting and lighting; permits, hazardous materials, environmental clearance; and traffic management.

Figure B-5 summarizes the BMP siting procedure for infiltration devices for all Districts except District 7, for which the procedures in Figure B-20 (page B-69) apply.

Figure B-4: Caltrans' Pilot Infiltration Trenches

Figure B-5: BMP Siting Procedure for Infiltration Devices



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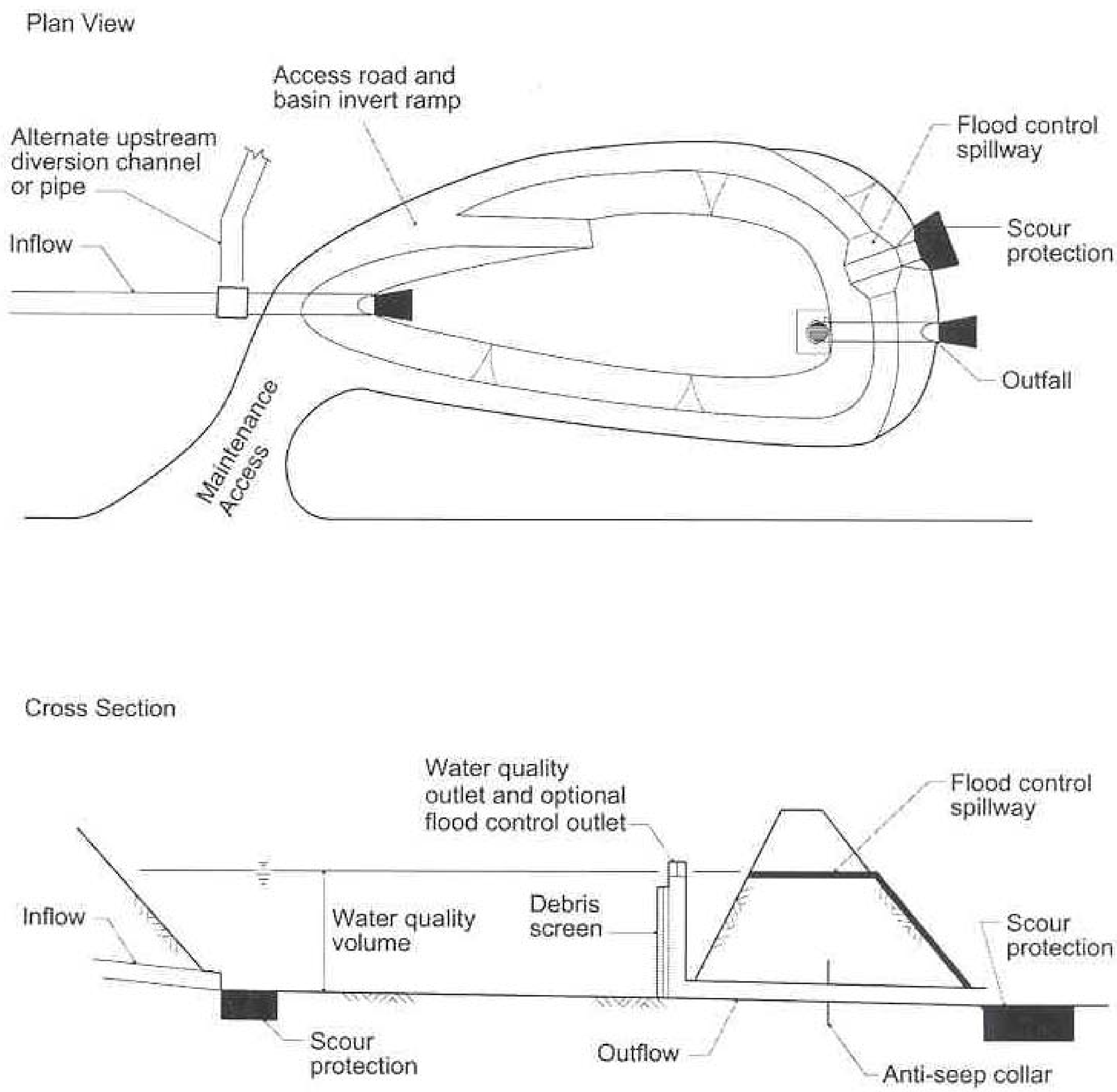
B.4 DETENTION DEVICES

B.4.1 Description

A detention device is a permanent treatment BMP designed to reduce the sediment and particulate loading in runoff from the water quality design storm (Water Quality Volume [WQV]). While the WQV is temporarily detained in the device sediment and particulates settle out under the quiescent conditions prior to the runoff being discharged. A detention device is typically configured as a basin. A schematic of a detention basin is shown in Figure B-6.

Figure B-6: Schematic of a Detention Basin
See Note 7



7 Low flow channel is not shown.

Detention devices remove litter, total suspended solids (TSS), and pollutants that are attached (adsorbed) to the settled particulate matter.

B.4.2 Appropriate Applications and Siting Criteria

Detention devices should be considered for implementation wherever infiltration devices are not feasible as part of the process of considering other approved Treatment BMPs, the WQV is at least 123 cubic meters (0.1 acre-foot), and site conditions allow.

One important siting requirement is that sufficient hydraulic head is available so that water stored in the device does not cause an objectionable backwater condition in the storm drain system, which would adversely impact its ability to convey design storms as required in the HDM. A second siting requirement is that seasonally high groundwater cannot be higher than the bottom elevation of the basin for reasons described in the section below.

B.4.3 Factors Affecting Preliminary Design

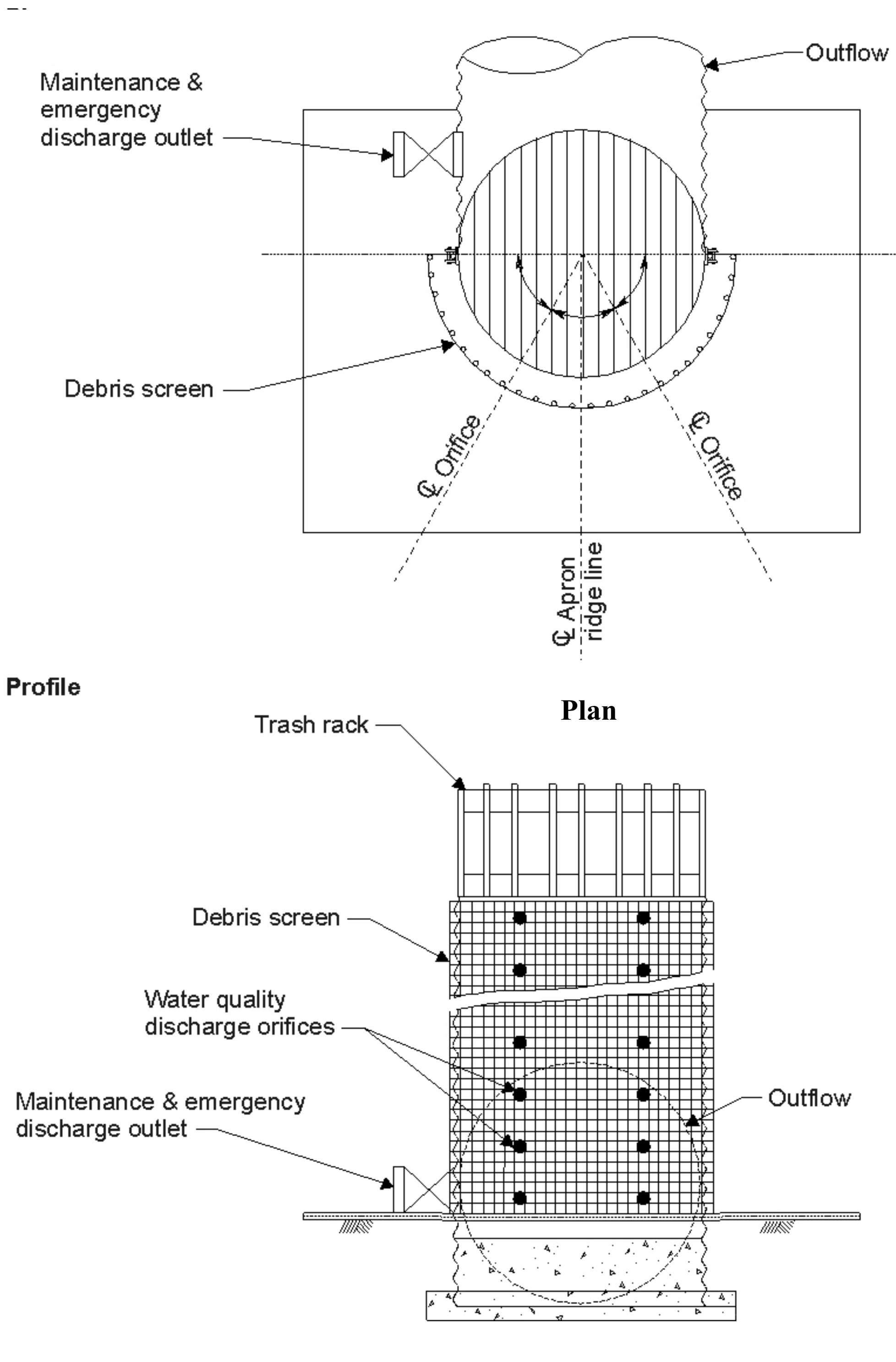
Detention basins should be designed with a volume equal to at least the WQV determined using the methods described in Section 2.4.2.2, Treatment BMP Use and Placement Considerations. The maximum water level in the detention device should not cause seepage of water under the roadway to within 0.2 m (8 in) of the roadway subgrade. The flow-path-to-width ratio within the detention basin at the elevation of the WQV is recommended to be ≥ 2:1; if needed, this ratio can be accomplished by baffles or interior berms to accommodate the geometry of the site.

Liners are not generally required for detention basins. However, they may be used to facilitate maintenance and to protect groundwater. Infiltration is permissible if the infiltrated water does not surface in an undesirable place off-site or threaten the stability of a slope or embankment downgradient of the basin. However, to protect groundwater quality and to ensure dry conditions for maintenance of unlined basins, the distance between the basin invert and seasonally high groundwater should be at least 3 m (10 ft); use a liner for an earthen detention basin if the groundwater separation distance between the basin invert and seasonally high groundwater is between 0.1 and 3m. In no case should the seasonally high groundwater be higher than the bottom elevation of the vault of a detention device structure, nor higher than the elevation of the liner for an earthen detention basin, in order to prevent uplift.

Entering flows should be distributed uniformly at low velocity to prevent re-suspension of settled materials and to encourage quiescent conditions. Low flow channels are often used to limit erosion during low flows.

Discharge should be accomplished through a water quality outlet. An example is shown in Figure B-7 (page B-29). A rock pile or rock-filled gabions can serve as an alternative to the debris screen around the outlet, although the designer should be aware of the potential for extra maintenance involved should the pore spaces in the rock pile clog. Proper hydraulic design of the outlet is critical to achieving good performance of the detention basin. The water quality outlet should be designed to empty the device within 24 to 72 hours (also referred to as "drawdown time"). The 24-hour limit is specified to provide adequate settling time; the 72-hour limit is specified to mitigate vector control concerns.

Figure B-7: Schematic of Water Quality Outlet Structure



Profile

The two most common outlet problems that occur are: a) the capacity of the outlet is too great resulting in only partial filling of the basin and drawdown time less than designed for; and b) the outlet clogs because it is not adequately protected against trash and debris. To avoid these problems, the following outlet types are recommended for use: (1) a single orifice outlet with or without the protection of a riser pipe[8], and (2) riser perforated vertically (orifices in multiple rows). Use of a V-notch weir as an outlet is not recommended because this design is susceptible to clogging. Design guidance for single orifice and for a perforated riser outlets is below.

Flow Control Using Orifices At The Bottom Of The Basin: The outlet control orifice should be sized using the following equation:

$$a = \frac{2A(H-Ho)^{0.5}}{3600CT(2g)^{0.5}} = \frac{(7x10^{-5})A(H-Ho)^{0.5}}{CT} \qquad \text{(Eq. 6)}$$

where:

a	=	total area of orifice (^{m2} or ft^2) (See Footnote 10)
A	=	surface area of the basin at mid elevation (m^2 or ft^2)
C	=	orifice coefficient (see discussion on following page)
T	=	drawdown time of full basin (hrs)
g	=	gravity (9.82 m/s^2 or 32.2 ft/s^2)
H	=	elevation when the basin is full (m or ft)
Ho	=	final elevation when basin is empty (m or ft)

With a drawdown time of 40 hours, the equation becomes:

$$a = \frac{(1.75x10^{-6})A(H-Ho)^{0.5}}{C} \qquad \text{(Eq. 7)}$$

For a riser perforated vertically (orifices in single or multiple columns (see Figure B-7), use:

$$a_t = \underline{[2A \times h_{max})]}/[3600 \times C \times T(2g\{h_{max} - h_{centroid\ of\ orifices}\})^{0.5}] \qquad \text{(Eq. 8)}$$

with terms as shown in Eqn. 7 except:

a_t	=	total area of orifices in the perforated riser, (m^2 or ft^2);
h_{max}	=	maximum height from lowest orifice to the maximum water surface (m or ft);
$h_{centroid\ of\ orifices}$	=	height from the lowest orifice to the centroid of the orifice configuration (m or ft).

Allocate the orifices evenly on two rows; separate the holes by 3x hole diameter vertically, and by 120 degrees horizontally.

If the WQV (specifically Options 1 and 2 in Section 2.4.2.2, Treatment BMP Use and Placement Considerations) was determined using an assumed drawdown time, then use the same value for

[8] In the 'single orifice' design, the total orifice area is placed at one elevation, and may be configured using one or several orifices, at the designer's option.

drawdown time (T) in equations 2 and 4. Because detention basins are not maintained for infiltration, water loss by infiltration should be disregarded when designing the hydraulic capacity of the outlet structure.

Assuming an average release rate at one half the basin depth (a common approach in several design manuals) may lead to considerable error if the basin has a significant variation of surface area with depth. If this is true, consult HEC-22, Chapter 10, for the design of detention facilities.

Care must be taken in the selection of "C"; 0.60 is most often recommended and used. However, based on actual tests, GKY (1989), "Outlet Hydraulics of Extended Detention Facilities for Northern Virginia Planning District Commission", recommends the following:

C = 0.66 for thin materials; where the thickness is equal to or less than the orifice diameter, or

C = 0.80 when the material is thicker than the orifice diameter

Drilling the orifice into an outlet structure that is made of concrete can result in considerable impact on the coefficient, as does the beveling of the edge.

Three alternative outlet structures that use single orifice outlets may be considered: a) A concrete block structure located in the containment berm for large basins. b) A riser pipe for small to large basins to prevent orifice clogging as shown in the equations above. c) Placing the outlet control downstream of the facility in the berm or in a manhole located may be considered for small basins as long as other outlets/spillways are provided for storms larger than the water quality design storm (consult District Hydraulics). For small facilities, place the control orifice in the outlet manhole downstream of the filter, or use a "T-pipe" to submerge the orifice. Variations of this alternative may include gates, valves, or weirs. The PE should consult with both the District Maintenance Storm Water Coordinator and the District Hydraulics Branch regarding these outfall structures.

Flow Control Using the Perforated Riser: For outlet control using the perforated riser as the outlet control, as shown on Figure B-7 (page B-29). This design incorporates flow control for the small storms in the perforated riser, and also provides an overflow outlet for large storms. If properly designed, the perforated riser can be used for both water quality and emergency overflow control by: (1) sizing the perforated riser as indicated for water quality control; (2) sizing the top of the outlet riser pipe to function as an overflow weir to control peak outflow rate from the from largest design storm that can enter the basin (up to the 100-year storm).

If possible, the inlet structure of the basin should be designed to divert the peak hydraulic flow (calculated according to Caltrans procedures for flood routing and scour) when the basin is full. Alternatively, an overflow structure sized according to these criteria can be provided in one of the downstream walls or berms. A third alternative is to include a flood control outlet in the top of the water quality outlet, as described in a preceding paragraph. In this case, an additional outlet (riser or spillway) is often still supplied to prevent overtopping of the walls or berms should blockage of the riser occur, based on a downstream risk assessment.

A detention basin must be designed to allow for regular maintenance. Consideration should be made for a perimeter access road, safe access to and from the site from local streets or access roads, and an access ramp to the basin invert. Any diversion from these requires the concurrence from the Maintenance Storm Water Coordinator.

Public health and vector control authorities should be consulted to verify the acceptability of detention basins and to establish the maximum drawdown time allowed in order to avoid mosquito problems.

Preliminary design factors for detention basins are summarized in Table B-4. A detention basin designed for dual purposes of water quality and flood control/attenuation requires additional design considerations not included in this table.

Detention basins will appear more aesthetic to the traveling public and function more effectively if vegetated on the invert rather than having a 'hard bottom' (outside of the low-flow paved area) and sideslopes; this will also eliminate the erosion from the sideslopes of the basin. [9] Consult the District Office of Landscape Architect for types of vegetation that can function effectively in Detention Basins in each of the various ecological subregions of a District. Additional information about grasses that have been successful within specific ecological subregions of California, in grassland and wetland conditions, may be found in Ecological Subregions of California Section and Subsection Descriptions (as referenced in Appendix B, Biofiltration Strips and Swales).

[9] If a vegetated invert is used, consider adding a paved ditch between the influent pipe and the outlet device, to reduce erosion caused under the initial flows into the basin.

Table B-4: Summary Of Detention Device Siting And Design Criteria

Description	Applications/Siting	Preliminary Design Factors
Impoundments where the WQV is temporarily detained during treatment Treatment Mechanisms: • Sedimentation • Infiltration (if basin unlined) Pollutants primarily removed: • Sediment (TSS) • Particulate metals • Litter • Sorbed pollutants (heavy metals, oil and grease [O&G]) to some degree	• WQV $\geq$ 123m^3 [0.1 acre-feet] • Sufficient head to prevent objectionable backwater condition in the storm drain system • Separation between seasonally high groundwater and basin invert > 3m; use liner if separation between 0.3m and 3m. • Consult public health and vector control authorities • Minimum orifice size of 13 mm (0.5 in) • If significant sediment is expected (e.g., from erosion-prone cut slopes) consider increasing the volume of the detention device an amount equivalent to the annual loading (or more, if less frequent cleanout is expected); consult with District Maintenance. • Locate outside the 9 m (30 ft) Clear Recovery Zone, or consult with Traffic Operations to determine if guard railing is required	• Size to capture the WQV according to Section 2.4.2.2. • Outlet designed to empty basin within 24 to 72 hrs (consistent with device sizing method), with 40 to 48 hours recommended, using debris screen (or equivalent). • Flow-path-to-width ratio of at least 2:1 recommended. • Maximum water level should not cause groundwater to occur under the roadway within 0.2 m (0.7 ft) of the roadway subgrade. • Maintenance access (road around basin and ramp to basin invert). • Upstream diversion channel or pipe (see Note 1), if possible. • Downstream spillway or overflow riser: sized to pass the largest storm (up t the 100-yr storm) that can enter the basin; minimum spillway length of 1 m, and/or minimum riser diameter of 900 mm (36 in.), or per District practice. Use local criteria for emergency flow passage if more stringent. • Provide freeboard ≥ 300 mm (12 in) (distance between the elevation of water in the basin when passing the largest storm that can enter the device and the elevation at the top of the confinement. • Provide an emergency/maintenance gravity drain, 200 mm diameter (8 inches) connected at the base of the outlet riser. • Flows should enter at low velocity. Use scour protection on inflow, outfall and spillway if necessary; a low flow channel may be used within the basin. • Use 1:4 slope ratios or flatter for interior slopes, unless approved by District Maintenance, with 1:3 maximum. • Provide vegetation on (earthen) invert and on non-paved side slopes, for performance and aesthetics.

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B.5 TRACTION SAND TRAPS

B.5.1 Description

Traction sand traps are sedimentation devices that temporarily detain runoff and allow traction sand that was previously applied to snowy or icy roads to settle out. In this handbook, traction sand refers to sand and other abrasives. These traps may take the form of basins, tanks, or vaults.

B.5.2 Appropriate Applications and Siting Constraints

Traction sand traps should be considered at sites where sand or other traction-enhancing substances are commonly applied to the roadway. If sand is used only rarely (less than twice a year), traction sand traps need not be considered for installation.

Vault-style traction sand traps should be considered only where detention basins or basin-style sand traps are infeasible.

The local RWQCB should be consulted by the District/Regional NPDES coordinator to ensure that the traction sand trap, is not classified as a regulated underground injection well.

B.5.3 Factors Affecting Preliminary Design

Traction sand traps are sized to convey the design peak flow while holding one year's worth of traction sand (or some other period of time chosen by the District). However, provisions should be made to divert the peak hydraulic flow (calculated according to the Caltrans procedures for flood routing and scour) if possible. Traction sand traps should have sufficient volume to store the settled sand with enough depth over the stored sand to prevent scouring and to promote relatively calm pool conditions.

The volume required to store traction sand is calculated by starting with the estimated amount of traction sand spread in a tributary area and applying reduction factors to account for sand that has been recovered by other means or that cannot be captured. The equation for calculating the volume of traction sand storage is:

$$V = (S \times R \times L \times E)/F \qquad \text{(Eq. 9)}$$

where:

V = The total volume of traction sand that must be stored (m^3).
S = The estimated volume of sand applied (m^3/yr).
R = A factor to account for sand recovered by roadway sweeping.
L = A factor to account for other miscellaneous losses/accumulations.
E = A factor to account for recovery efficiency.
F = The number of times the trap will be cleaned (times/yr).

Guidelines for defining the variables in this equation are as follows:

S: Typical sand application rates range from 47 m^3/lane/km/yr for areas with average application rates to 95 m^3/lane/km/yr for areas with high application rates. To estimate the total volume of traction sand applied, select an appropriate application rate from the range listed in this section, and multiply it by the total number of lanes (e.g., one lane in each direction equals two lanes) and the length of highway tributary to the sand trap. Because some areas track sand usage by post mile, a more accurate estimate may be obtained by consulting with District maintenance staff. In any event, consider the following guidelines when estimating the volume of sand that is spread annually in the tributary area:

Exposure: Roadways on north facing slopes generally require more traction sand than similar south facing slopes. The surrounding vegetation may also significantly affect exposure and traction sand application.

Roadway grade: Steeper grades generally receive more traction sand than flatter grades.

Other climatic and geographic factors, such as elevation, will affect the traction sand application rate for a specific area.

Other sources of similar material: Adjacent cut slopes and other non-paved tributary areas may contribute similar-sized sediment or other debris that will be retained in the trap.

R: This is a factor to account for traction sand that is recovered through roadway sweeping. Estimate a value between 1.0 (no roadway sweeping) and 0.6 (aggressive winter roadway sweeping) based on interviews with District maintenance staff. If actual sweeping records are available, these may provide a more accurate estimate.

L: This is a factor to account for traction sand that has been carried into or out of the tributary area by miscellaneous means such as wind (smaller particles), sand thrown out of the tributary area by snow clearing equipment, and sand splashed or carried by vehicles. Estimate an appropriate value in the range of 0.8 (high losses from known sources such as snow blowers) to 1.2 (high accumulation from known sources). Use a factor of 1.0 for no miscellaneous losses/accumulations.

E: This factor is provided to account for traction sand that passes through the sand trap without settling out. Because of particle size limitations, settling inefficiencies, and other factors, it may not be realistic or practicable to recover all of the traction sand that reaches the sand trap. Until empirical information is obtained from pilot studies, a value of 1.0 should be used for this factor.

F: This is the number of times the sand trap will be cleaned each year. Usually, the value for F is 1 as most basins are cleaned once per year, usually in the summer. If obtaining the required storage volume is difficult, it may be possible to implement mid-season cleaning (F greater than 1), but District maintenance staff should be consulted to make sure this is practicable. Mid-season cleaning requirements will also likely affect trap design, as maintenance equipment will have to access the trap under wet or snowy conditions.

Other design issues: Traction sand traps configured as vaults require a small hydraulic head for gravity flow operation. The inlet and outlet devices should be arranged or baffled to minimize short-circuiting of the flow through the device. In single cell tanks and vaults, provide if possible at least 150 mm between top of captured sand and outlet pipe. Weep holes should be provided and the trap invert should be sufficiently high above groundwater (1 to 2 m) (3 to 6 ft) to allow for proper drainage. Traction sand traps that do not drain may create vector problems in the spring.

Maintenance needs: Traction sand traps require sufficient space and/or access ramps for maintenance by large equipment to remove the accumulated sand. Traps should also be located so that water is not infiltrated above the roadway subgrade should the trap become blocked or fail to drain so as not to affect expected life of the pavement.

Preliminary design factors for traction sand traps are summarized in Table B-5.

Table B-5: Summary of Traction Sand Trap Siting and Design Criteria

Description	Applications/Siting	Preliminary Design Factors
Sedimentation devices that temporarily detain runoff and allow traction sand to settle out. May be basins, tanks, or vaults. Designed for peak hydraulic flow. Treatment Mechanisms: • Sedimentation Pollutants removed: • Sand or other traction-enhancing substances	• Sites where sand or other traction-enhancing substances are commonly applied to the roadway • Not considered where sand is used only rarely (less than twice a year) • Use detention basins or forebays as traction sand traps whenever feasible; if they are not feasible, then consider tanks or vaults • Consult District/Regional NPDES Storm Water Coordinator to ensure device not classified as a regulated underground injection well • Locate device so water is not introduced above the roadway subgrade in case of blockage	• Design for anticipated sand recovery and cleanout interval • In the tributary area: minimize unstabilized areas that will contribute sediment as much as possible • Divert peak hydraulic flow if practical • Sufficient volume to store the settled sand through the winter and avoid scour • In single cell tanks and vaults, provide if possible temporary storage volume (for sedimentation) using a minimum of 150 mm between top of sand (just prior to scheduled cleanout) and outlet pipe • Sufficient hydraulic head for gravity flow • Inlet and outlet arrangement to minimize short-circuiting of the flow • Weep holes to allow proper drainage • Invert 1 to 2 m (3 to 6 ft) above groundwater if drainage is allowed through base (CMP riser type) • Maximum depth of tank or vault of 3 m below ground surface (varies with equipment – consult District Maintenance) • Maintenance space and/or access ramps for large equipment (a maintenance vehicle access shoulder of up to 4.9 m (up to 16 ft) may be required; consult with District Maintenance)

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B.6 DRY WEATHER FLOW DIVERSION

B.6.1 Description

Dry weather diversion flow devices provide permanent treatment by directing non-storm water flow through a pipe or channel to a local municipal sanitary sewer system (publicly owned treatment works [POTWs]) during dry season or weather. This flow must be generated by Caltrans activities or from Caltrans facilities.

B.6.2 Appropriate Applications and Siting Criteria

Dry weather flow diversion should only be considered when all of the following conditions apply:

- Dry weather flow is persistent (i.e., present over a significant length of time at a relatively consistent flow rate, or having significant quantities that are periodically developed on-site), and contains pollutants;
- An opportunity for connecting to a sanitary sewer is reasonably close and would not involve extraordinary plumbing to implement (e.g., jacking under a freeway);
- The POTW is willing to accept the flow during the dry season or weather.

An example of dry weather flow that could be considered for diversion is the runoff from a Caltrans tunnel generated during cleaning using water spray and scrubbing, since the wash water typically will contain soot.

Flow from outside of the right of way may be considered for a dry weather flow diversion if the following conditions are met:

- Dry weather flow is persistent (i.e., present over a significant length of time at a relatively consistent flow rate, or having significant quantities that are periodically developed on-site);
- An opportunity for connecting to a sanitary sewer is reasonably close and would not involve extraordinary plumbing to implement (e.g., jacking under a freeway); and
- Dry weather flow diversion of the flow crossing the right of way is recommended by local health officials because of the detriment to the beneficial us of the downstream receiving water.

B.6.3 Factors Affecting Preliminary Design

Typically, a berm or wall is constructed across the dry weather flow drainage channel and the dry weather flows are diverted to a pipe or channel leading to the sanitary sewer. A gate, weir, or valve should be installed to stop the diversion during the wet season or during storms during the wet season (if the diversion will be made year-round). Accordingly, the conveyance to the sanitary sewer should be sized for the dry weather (non-storm) flows only. Wet weather flow is diverted (or remain undiverted, depending upon the design) back to the storm water conveyance system.

If possible, a screen should be installed at the diversion to reduce the likelihood of clogging the diversion pipe or channel. Maintenance vehicle access should be provided, especially if a screen is installed.

Preliminary design factors for dry weather flow diversions are summarized in Table B-6.

Table B-6: Summary of Dry Weather Flow Diversion Siting and Design

Description	Applications/Siting	Preliminary Design Factors
Direct flow during dry weather (or non-storm periods) to a POTW. Treatment flow rate determined on a site-specific basis (not the WQF). Treatment Mechanisms: • Wastewater treatment plant Pollutants removed: • All constituents	Only when the conditions below apply: • Dry weather flow is persistent (consistent flow rate and significant length of time) • Connection would not involve extraordinary plumbing to implement • POTW willing to accept dry weather flow	• Berm or wall across channel to divert dry weather flow to the sanitary sewer • Gate, weir, or valve to stop diversion during wet season • Conveyance to sanitary sewer sized only for dry weather flow • Consider a screen to limit debris conveyed to the POTW • Maintenance vehicle access

B.7 GROSS SOLIDS REMOVAL DEVICES: LINEAR RADIAL DEVICE AND INCLINED SCREEN DEVICES

B.7.1 Description

Gross Solids Removal Devices (GSRDs) include physical or mechanical methods to remove litter and solids 5 mm (0.25 inch nominal)[10] and larger from the storm water runoff, usually done using various screening technologies. GSRDs should be considered for projects in watersheds where a TMDL allocation or 303(d) listing for litter has been made. The design should be coordinated through the Headquarters – Office of Storm Water Management – Design. GSRDs should be designed to handle up the HDM Section 800 storm event, typically Q_{25}, unless placed in an off-line configuration. The devices also have an emergency overflow capacity in the event of clogging.

B.7.2 Appropriate Applications and Siting Criteria

There are currently two approved types of GSRDs that can be considered:

- The Linear Radial – this device requires very little head to operate and is well suited for narrow and relatively flat rights-of-way.
- The Inclined Screen – this device requires about 1.5 m (5 ft) of head and is better suited for fill sections of the highways.

GSRDs require sufficient space and/or access ramps for maintenance and inspection including the use of vacuum trucks or other large equipment to remove accumulated trash.

B.7.3 Styles of Devices

B.7.3.1 Linear Radial Device

The Linear Radial Device (Figure B-8, page B-42) utilizes modular well casings with 5 mm (0.25-inch nominal) louvers to remove litter. The louvered well casings are contained in a concrete vault. Flows pass radially through the louvers trapping litter and solids in the casing and passing flows into the vault for discharge via an outlet pipe. The bottom of the casing is smooth to allow trapped litter to move to the downstream end of the well casing. The Linear Radial Device is designed to work in-line with the existing storm drain system or could be placed in an off-line configuration; either placement will incorporate an overflow/bypass that will operate if the unit becomes plugged. As shown in Figure B-8, the first half-meter of the linear well casing is non-louvered with an open top to allow for influent bypass should the device become clogged with litter. The circular louvered sections have access doors that can be easily opened to facilitate cleaning with a vacuum truck or other equipment if necessary.

[10] The 5 mm dimension is based on requirements set forth in TMDLs applicable to certain District 7 watersheds; other sizes may be necessary if required to meet TMDLs issued by other RWQCBs.

Figure B-8: Schematic of Linear Radial Device

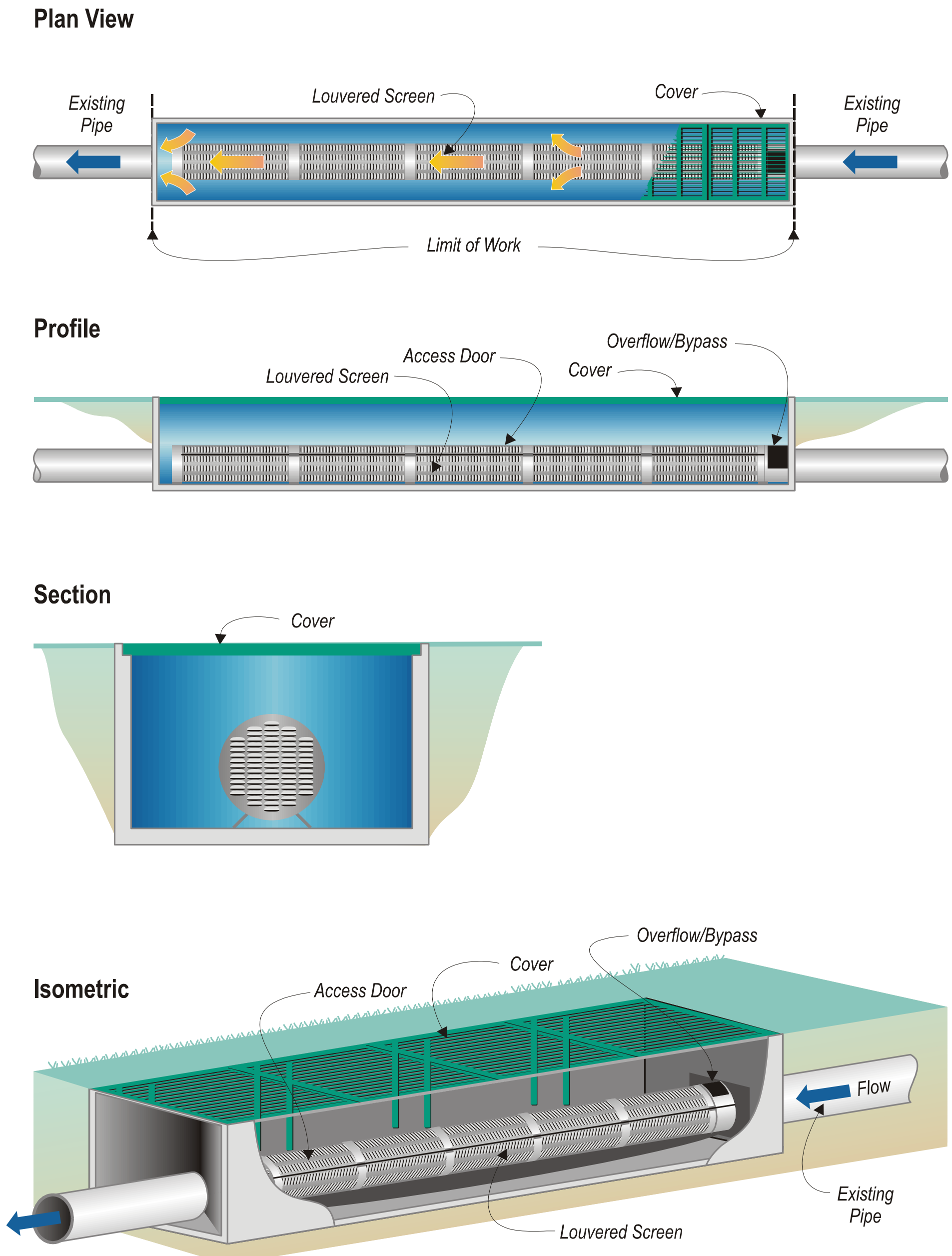


Figure B-9: Linear Radial Device (partially full)

B.7.3.2 Inclined Screen Devices

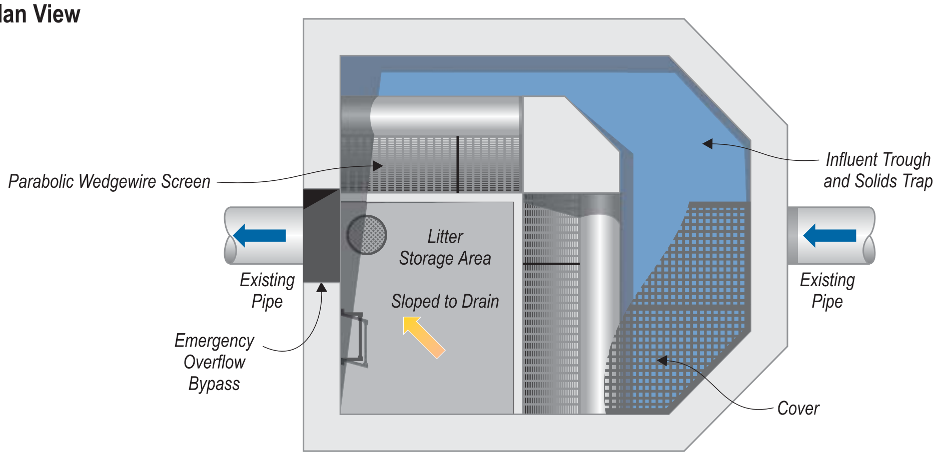
Two versions of the Inclined Screen Device have been tested. In one the incoming flow overtops a weir and falls through an inclined bar rack (wedge-wire screen) with a 3-mm (0.125-inch nominal)[11] maximum spacing between the bars, located after the influent trough. After passing through the rack, the flow exits the device via the discharge pipe. A distribution trough is provided to allow influent to be distributed along the length of the Inclined Screen. The litter captured by the bar rack is pushed down toward the litter storage area by the storm water runoff. This version employs a parabolic wedge-wire screens inclined at 60 degrees and 1 m (3 ft) high. The gross solids storage area is sloped and is provided with a drain to prevent standing water. As shown in Figure B-10 (page B-44), an opening above the litter storage area is provided to allow for overflow/bypass if the device becomes plugged. The device should be designed for litter and debris storage for a period of one year.

A second version uses a straight screen, and incoming flow is not required to overtop a weir to reach the screen (figure not provided).

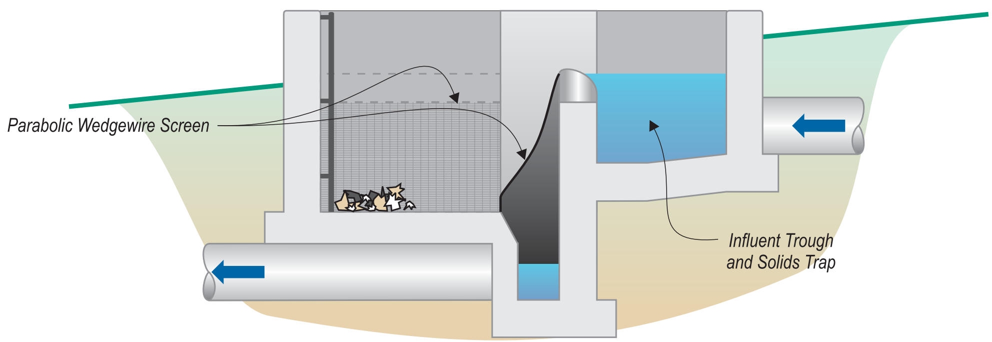
[11] This screen size was pilot tested; other screen sizes up to 5 mm (0.25 inch) may be used if available.

Figure B-10: Schematic of Inclined Screen Device [12]

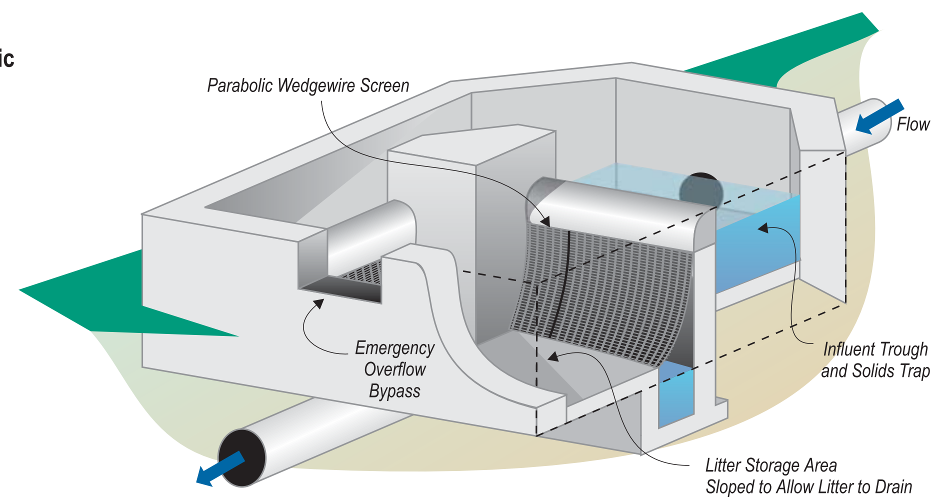
Plan View



Profile



Isometric



[12] Only the parabolic screen is shown.

Figure B-11: Inclined Screen GSRD

B.7.4 Factors Affecting Preliminary Design

The two most important factors affecting the design of these devices are: a) the need to be sized to accommodate both gross pollutants storage for a given maintenance period (typically one year), and b) the hydraulic capacity of the drainage system in which it is to be installed. Litter and debris accumulation data need to be available to properly size the devices for the given drainage area. If regional debris accumulation data are not available, then 0.7 m^3/ha/yr may be used and consult with District Maintenance. These devices can be designed both in-line and off-line.

A summary of preliminary design factors is presented in Table B-7.

Table B-7: Summary of Gross Solids Removal Devices (Linear Radial and Inclined Screen)

Description	Applications/Siting	Preliminary Design Factors
Devices to capture and remove litter from the storm water runoff. • Designed to handle up the HDM Section 800 storm event, typically Q_{25} unless placed in an off-line configuration Treatment Mechanisms • Filtration through screens Pollutants removed • Litter and solid particles greater than 5 mm (0.25 inch nominal)	• Site conditions must have adequate space for device and maintenance activities. • Sites that drain to litter sensitive receiving waters on 303(d) list for trash or areas where TMDLs require trash removal. • The Linear Radial Device requires little head to operate and is well suited for flat sections of highway. • The Inclined Screen requires 1.5 m (5 ft) of head measured between the top of the weir above the screen and the flowline of the outflow pipe; it is well suited for fill sections. • Locate outside the 9 m (30 ft) Clear Recovery Zone, or consult with Traffic Operations to determine if guard railing is required	• Design using regional litter accumulation data is desirable, otherwise use 0.7 m^3/hectare/year. • Devices must be sized for peak design flow while holding design gross solids load. • Some TMDLs also require full capture for events of up to a one-year, one-hour storm event (i.e., runoff should not be bypassed in the GSRD under that flow rate). Determine if this or other specific TMDL requirements apply at the project site. • The Linear Radial Device well casing is available up to 900 mm (36 inch) diameter, but a special design is required. • Divert peak hydraulic flow if possible; devices can be placed in-line with a overflow device, or off-line of the drainage system by using a bypass device (consult with District Hydraulics • Consider need for traffic rated access grates (depending upon placement location of device) • Determine location and depth of device for maintenance access (coordinate with District Maintenance)

B.8 MEDIA FILTERS

B.8.1 Description

A Media Filter Treatment BMP device primarily removes TSS pollutants (sediments and metals) from runoff by sedimentation and filtering, and also is effective for dissolved metals and litter.

There are two types of approved Media Filter devices: The Austin Sand Filter and the Delaware Filter; each is configured using two chambers. An 'Austin' sand filter is usually open and at grade, and has no permanent water pool; a 'Delaware' sand filter is always configured with closed chambers and below grade, and has a permanent pool of water. An Austin sand filter may be configured with earthen sides and invert, but usually has chambers made using concrete; a Delaware sand filter is always made using concrete sides and invert.

In both types of media filters, storm water is directed into the first chamber where the larger sediments and particulates settle out, and the partially treated effluent is metered into the second chamber to be filtered through a media. In the Austin sand filter, the first chamber may be sized for the entire WQV ('full sedimentation') (see Figure B-13, page B-48) or as a 'partial sedimentation' chamber, holding only about 20% of the WQV (see Figure B-14, page B-49); the Delaware sand filter holds the entire WQV in the initial chamber, and is designed to pass the WQV from the second chamber (see Figure B-15, page B-50).

The treated effluent (filtered water) is captured by perforated underdrains (collector pipes) for release downstream. There is a drop in elevation of 0.9 m to 1.2 m between the invert of the first and second chambers.

The filter media typically consists of sand, which is effective for removal of coarse and fine sediments and particulate metals. Other materials, such as topsoil or organic materials, may be added to the sand to increase the treatment capacity for some pollutants (for example, dissolved metals) but these additives often increase the nitrogen and phosphorus concentration levels in the effluent. Design of a wet basin must be coordinated through the Headquarters Division of Environmental Analysis – Policy, Planning and Permitting, and Headquarters Design – Office of Storm Water Management.

Figure B-12: Caltrans Pilot Media Filters

B.8.2 Appropriate Applications and Siting Criteria

The minimum WQV for Media Filters is ≥ 123 m^3 (0.1 acre-ft [a-f])[13]. Medial Filters will perform better if the tributary area has a relatively high percentage of impervious area, and low sediment loading.

Sites proposed for Media Filters must have sufficient hydraulic head to operate by gravity; about 0.9 to 1.8 m (3 to 6 ft) is needed between the inflow to the initial chamber and effluent outflow from the second chamber.

Placement of the Delaware Filter should be avoided in locations where there are concerns about vectors because they maintain a permanent pool of water, unless concurrence for its use can be obtained from the local vector control agency.

At least 1 m (3.3 ft) separation from seasonally high groundwater for the initial chamber (if soft-bottomed); the second chamber (vault) may be at or below seasonally high groundwater if waterproof joints are specified for the pipe carrying the treated effluent and uplift does not occur.

Inflows in cold regions may not be treated in the second chamber due to freezing, unless below frost line.

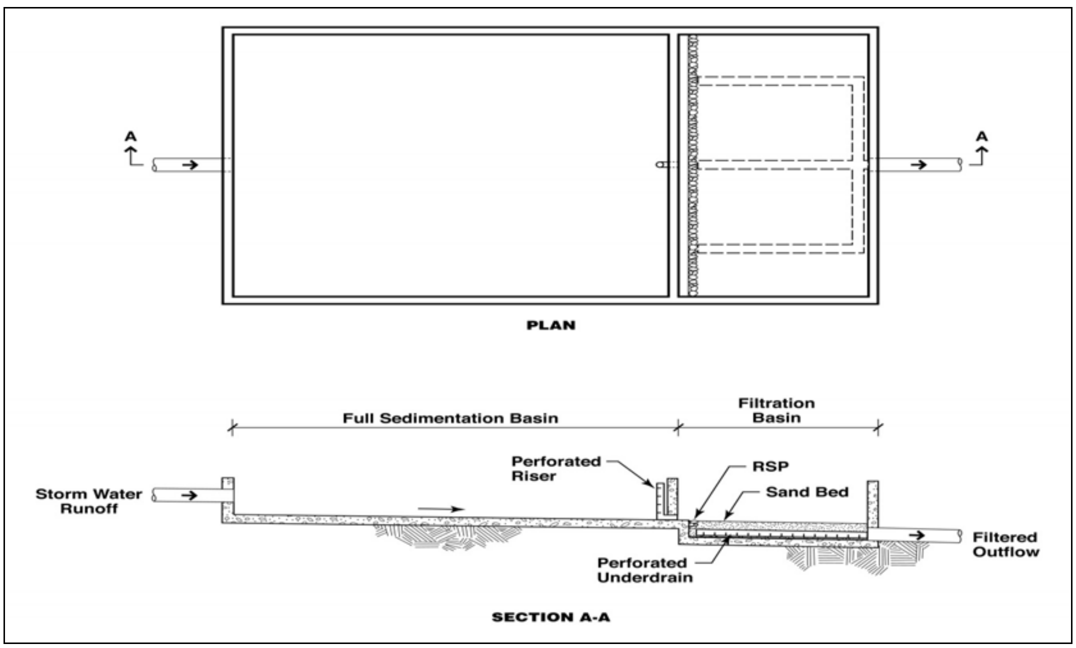


Figure B-13: Schematic of the Austin Sand Filter (full sedimentation version)

[13] Consult with District/Regional NDPES if less than 123 m^3 is under consideration.

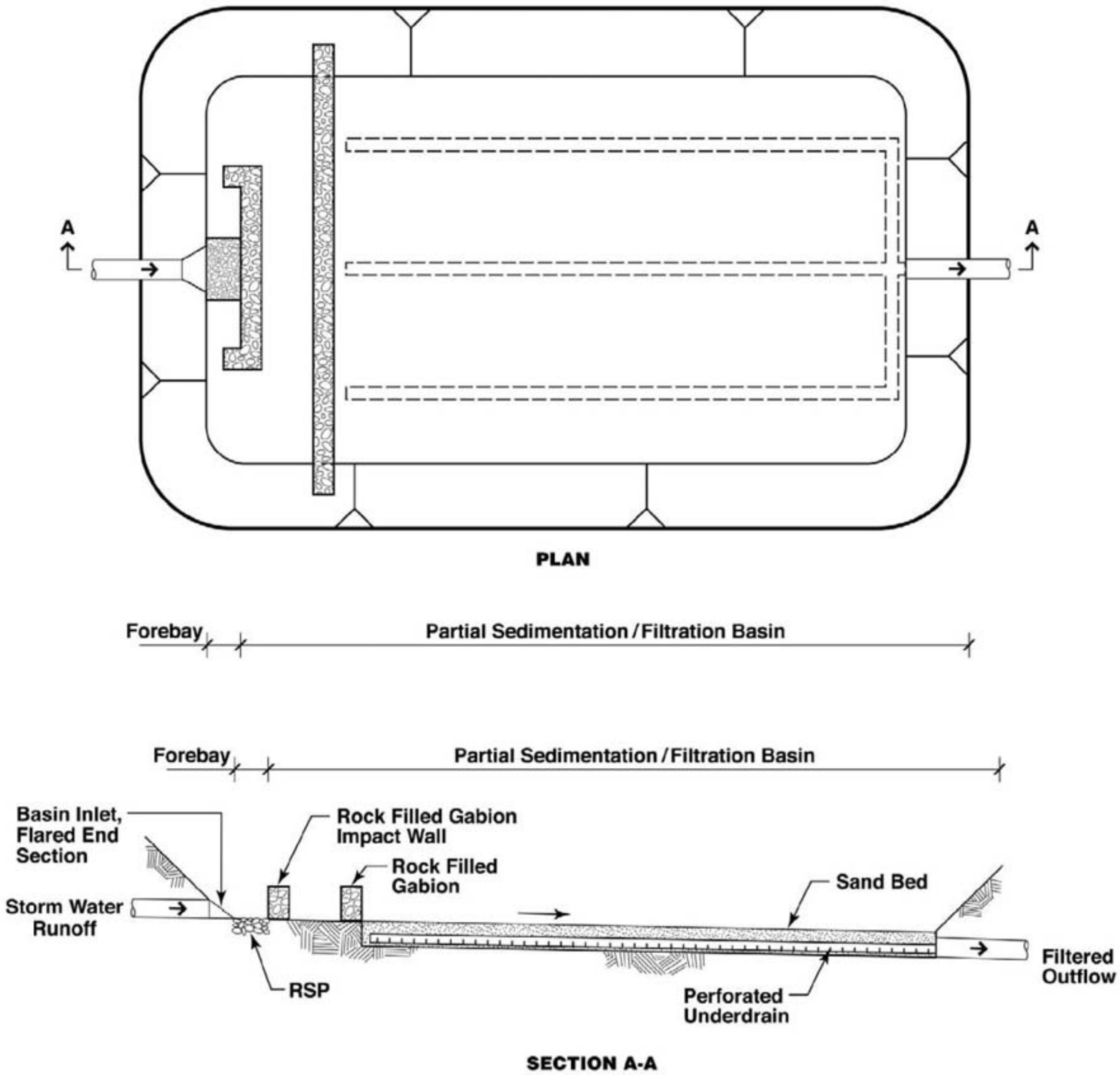


Figure B-14 Schematic of an Austin Sand Filter Partial Sedimentation Device

B.8.3 Preliminary Design Factors

B.8.3.1 General Factors

Maintenance must have access to both chambers, and the distance below ground surface of the invert must be approved by Maintenance (maximum depth of 4 m [13 ft]).

Austin, full sedimentation design should have the following design features: a) the initial chamber should be sized to hold the entire WQV at a 24-hour release time; b) release to the second chamber is usually made using a perforated riser; and c) a desired length to width ratio of 2:1 should be provided.

For partial sedimentation designs the following features apply: a) the initial chamber should be sized to hold about 20% of the WQV; b) release from the first chamber is made using a rock-filled gabion wall separating the chambers; c) the length to width ratio does not apply; and d) the combined volume of the both chambers should be ≥ the WQV.

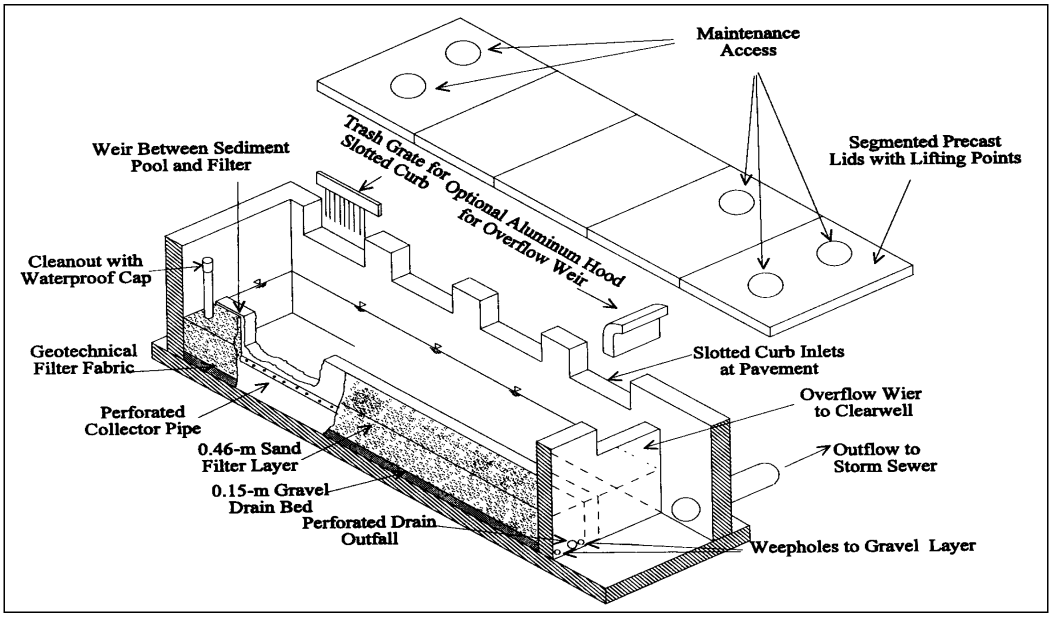


Figure B-15: Schematic of a Delaware Sand Filter (Young et al., 1996)

Austin: Depth of the media layer (sand filter layer) typically 0.46 m, and the gravel layer (collector layer) is 0.15 m.

Delaware: Depth of the media layer (sand filter layer) is 0.46 m; depths of the two gravel layers are: top layer at 50 mm, and lower layer (collector layer) at 0.40 m. Separate layers using geotextile fabric.

Austin Sand filter with earthen base and sides, full or partial: side slopes should be 1V:3H or flatter, and should be stabilized by vegetation.

Upstream bypass for larger storms is preferred for storms > WQV; internal overflow protection also must be provided through the device, typically using weirs from the initial chamber.

Upstream litter and sediment capture should be provided if possible, e.g., using biofiltration or a forebay.

Preliminary Design Factors for Media Filters are summarized in Table B-8.

B.8.3.1 Austin Sand Filter Chambers, full sedimentation device

Size the initial chamber to hold the WQV, and use the equation for the outlet riser presented under Detention Devices to determine the diameter of the orifices, using a 24-hour hold time.

The equation for sizing the filter bed in the second chamber is:

$$A_{ff} = [FS \times C \times WQV \times d)] / [k \times T \times (h + d)] \quad \text{(Eq. 10)}$$

where

A_{ff} = area of 2nd chamber filter bed, full sedimentation basin; m^2 or ft^2
FS = factor of safety; 2
C = conversion factor for units of permeability (100 for cm to m; 12 for inches to ft)
WQV = Water Quality Volume; m^3
d = depth of sand layer in the Austin-style filter bed, typically 0.46 m.[14] or 1.5 ft
k = coefficient of permeability of the filtering medium; cm/hr. metric: 10 cm/hr; US Customary units: 4 inches/hr
T = design drain time for WQV, typically 24 hours, but must be consistent with the first chamber
h = average water height above the surface of the media bed, taken as ½ the maximum head of the second chamber (distance to any overflow device from that chamber to the surface of the media bed); m or ft

B.8.3.2 Austin Sand Filter Chambers, partial sedimentation device

Sizing the two chambers for the Austin Sand Filter partial sedimentation device:

First, size the filter bed in the second chamber using the following formula:

$$A_{fp} = 1.8A_{ff} \quad \text{(Eq. 11)}$$

where A_{fp} is a partial sedimentation chamber, and A_{ff} is calculated as above.

Note that the filter area is larger in the partial sedimentation version than the full sedimentation version due to the less efficient capture of sediments in the partial sedimentation device.

Then size the initial chamber to hold a minimum 20% of the WQV, subject to increase to meet the requirement that both chambers (including the void space in the filter chamber calculated using Eqn. 15 shown on page B-53) combine to hold the entire WQV.

With these requirements, the area of the partial sedimentation Austin Filter is usually about 80 to 90% of the full sedimentation Austin Sand Filter. However, the efficiency of the partial sedimentation design is not greatly reduced from the full sedimentation version, and the maintenance is usually reduced because the release of storm water from the partial sedimentation chamber to the filter chamber is usually done through a rock-filled gabion wall (and not an outlet riser), and no hold time is assigned to the water in the initial (sedimentation) chamber.

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14 Note that in the final design a collector system must be placed below the Austin Media Filter, typically as an additional 0.15 m of gravel below the filter bed, and with perforated underdrains placed within at the bottom of this gravel layer. However, ignoring the correction for the effective vertical permeability of the stratified soil and gravel layers and the slight increase in depth of the combined layers introduces only minor error to the calculated area of the second chamber, given the large difference in permeability between sand and gravel, and can be ignored.

B.8.3.2 Delaware Sand Filter Chambers

Sizing the two chambers for the Delaware Sand Filter:

The area for the initial chamber (the sediment chamber, A_{sc}) is set equal to the area of the filter chamber (A_{fc}) and the area of the filter chamber is calculated using one of two formulas, depending upon the allowed depth of water in the sediment chamber:

If the maximum head of the initial chamber (2h) > 0.81 m:

$$A_{fc} = [FS \times C \times WQV \times d] / [k \times T \times (h + d)] \qquad \text{(Eq. 12)}$$

where

A_{fc}	=	area of filter chamber; m^2 or ft^2
FS	=	factor of safety; 2
C	=	conversion factor for units of permeability (100 for cm to m; 12 for inches to ft)
WQV	=	Water Quality Volume; m^3 or ft^3
d	=	depth of sand layer in the Delaware filter bed, typically: metric: 0.46 m.[15] US Customary units: 1.5 ft
k	=	coefficient of permeability of the filtering medium; metric: 5.0 cm/hr; US Customary units: 2 inches/hr
T	=	design drain time for WQV, typically 40 to 48 hours
h	=	average water height above the surface of the media bed, taken as ½ the maximum head of the second chamber (distance to any overflow device from that chamber to the surface of the media bed); m or ft

Note that the formula for the Delaware Sand Filter is very similar to that used for the full sedimentation Austin Sand Filter, except for the value assigned to the permeability (even though the same material is used); a more conservative permeability value is assigned to the Delaware Sand Filter, as the device, being underground and not directly visible during the wet season, requires a more conservative design. One other difference is that the 'h' term is measured in a different location.

If the maximum head of the initial chamber (2h) < 0.81 m and a 40-hour drawdown time is used:

$$A_{fc} = [100WQV \times d)] / [4.1h + d] \qquad \text{(Eq. 13)}$$

where all terms are as defined above.

[15] Note that in the final design for the Delaware Media, two gravel layers are placed: one is placed above the sand media, at 50 mm thickness, and the second layer, forming the collector system, is placed at 0.40 m below the filter bed; the perforated underdrains placed within lower gravel layer. However, ignoring the correction for the effective vertical permeability of the stratified soil and gravel layers and the slight increase in depth of the combined layers introduces only minor error to the calculated area of the second chamber, given the large difference in permeability between sand and gravel, and can be ignored.

Then, series of calculations must be made to verify that the required storage areas are sufficient. Step 1: Select a width for the chambers, normally between 0.46 and 0.76 m wide (18 to 30 inches, not including the concrete wall between them), and compute the length based on the area calculated above:

$$L_s = L_f = A/W \qquad \text{(Eq. 14)}$$

where
L_s = length of the sediment chamber, m or ft
L_f = length of the filter chamber, m or ft
A = Area used for both chambers (A_{sc} or A_{fc}), calculated above, m^2 or ft^2
W = selected width, m or ft

Step 2: Calculate the storage volume available for water in the filter chamber (filter media), V_V:

$$V_V = 0.35A_{fc} \times (d_f + d_g) \qquad \text{(Eq. 15)}$$

where
V_{fc} = effective volume of the filter chamber; m^3 or ft^3
A_{fc} = area of the filter chamber; m^2 or ft^2
d_f = depth of the filter (sand) layer; metric: 0.46 m; US Customary units: 1.5 ft
d_g = depth of the gravel layer(s); metric: 0.46 m; US Customary units: 1.5 ft
0.35 = assumed void ratio (dimensionless)

Step 3: Calculate the flow through the filter during filling, V_Q

$$V_Q = [k \times A_{fc} \times [d_f + d_g] \times t_f) / d_f \qquad \text{(Eq. 16)}$$

where
k, A_{fc}, d_f, d_g, and d_f are terms as defined above
t_f = time to fill the voids, take as 1 hour

Step 4: Calculate the net volume required to be stored in chambers awaiting filtration, V_{ST}

$$V_{ST} = WQV - V_V - V_Q \qquad \text{(Eq. 17)}$$

Step 5: Calculate available storage in chambers, V_{SF}

$$V_{SF} = 2h \times (A_fc + A_{sc}) \qquad \text{(Eq. 18)}$$

Step 6: Compare V_{SF} and V_{ST}

If $V_{SF} > V_{ST}$, proceed with the design

If $V_{SF} > V_{ST}$, adjust the length or the width or either chamber, and repeat these Steps 1 through 5.

Table B-8: Summary of Media Device Siting and Design Criteria

(Applicable to both Austin Sand Filter and Delaware Filter unless noted)

Description	Applications/Siting	Preliminary Design Factors
Two chambered treatment devices designed to treat the WQV. Treatment Mechanisms • Sedimentation • Filter Pollutants primarily removed • Suspended solids • Particulate metals • Dissolved metals • Litter (although preferred capture is upstream of the device)	• WQV ≥ 123 m^3 (0.1 a-f) • Site must have sufficient hydraulic head to operate by gravity between inflow to the initial chamber and effluent outflow from the second chamber, about 0.9 to 1.8 m (3 to 6 ft) • Delaware media filters should avoid locations where there are concerns about vectors because they maintain a permanent pool of water unless concurrence for its use can be obtained from the local vector control agency. . • Inflows in cold regions may not be treated in the second chamber due to freezing unless below frost line • At least 1 m (3.3 ft) separation from seasonally high groundwater for the initial chamber (if soft-bottomed); the second chamber (vault) may be at or below seasonally high groundwater if waterproof joints are specified for the pipe carrying the treated effluent and uplift does not occur • Will perform better if the tributary area has a relatively high percentage of impervious area, and low sediment loading • Maintenance must have access to both chambers • Locate outside the 9 m (30 ft) Clear Recovery Zone, or consult with Traffic Operations to determine if guard railing is required	• Maximum depth: 4 m below ground surface; verify with Maintenance • Upstream bypass for larger storms is preferred but bypass for storms > WQV must be provided through the device, typically using weirs from the initial chamber. • Provide if possible upstream litter and sediment capture, e.g., using biofiltration or a forebay • Collector pipes: minimum 150 mm (6 inches) diameter laterals, and minimum 200 mm (8 inches) diameter collector pipe • Sand media: use Caltrans Standard Specification 90-3.03 for fine aggregate; Gravel: use Caltrans Standard Specification 68-1.025, Permeable Material, Class 1, Type B; separate layers using geotextile.[16] • Austin, full sedimentation design: design the initial chamber to hold the entire WQV and use a 24-hour release time if site constraints allow, release to the second chamber using a perforated riser, and a length to width ratio of 2:1. For partial sedimentation designs, the initial chamber should be sized to hold ≥ 20% WQV, and both chambers must hold ≥ 100% WQV; a rock-filled gabion wall separating the chambers. For either: Drainage over 24 hours from the second chamber (filtering chamber) • Austin Sand Filter: no permanent vegetation is desired on the invert of the second chamber. • Austin Sand filter with earthen base and sides, full or partial: side slopes should be 1V:3H or flatter, and should be stabilized by vegetation.

[16] Media Filters: The filter fabric should meet the requirements of Caltrans Standard Specification Section 88-1.03, Filter Fabric. The gravel layer can function without an intermediary geotextile, if designed using 'graded filter' criteria (e.g., see Soil Mechanics, DM 7.01, NAVFAC, 1986, page 271ff).

B.9 MULTI-CHAMBER TREATMENT TRAIN (MCTT)

B.9.1 Description

The MCTT is a storm water treatment device that uses different treatment mechanisms in each of three sequential chambers. The MCTT was developed for treatment of stormwater at critical source areas, such as vehicle service facilities, parking areas, paved storage areas and fueling stations. A schematic of an MCTT is shown in Figure B-17 (page B-56).

The initial chamber, also called a 'grit' chamber, captures the larger sized sediments; this may be configured as a catch basin with a sump. Some variations are employed in this chamber, such as including a trash rack. The second chamber, also called the main settling chamber, is designed to capture finer sediments; this chamber may also be configured with sorbent pads or plates designed to capture hydrocarbons, and some designs employ aeration in this chamber (forcing air into the ponded water from approximately mid-elevation in the chamber) to lift floatables and litter not captured in the initial chamber. The third chamber, also called the filtering chamber, employs a media filter often configured as a combination of sand and peat moss; it removes even finer sized particles than were captured in the previous chambers, and acts as a sorption area for some dissolved constituents.

Figure B-16: Caltrans' MCTT pilot installations

Water flows from the initial chamber to the second chamber via either an overflow weir or an orifice, and this chamber will have a permanent pool of water. Water flows from the second to third chamber via either an orifice or a weir; if a weir used, the second chamber also would have a permanent pool of water. The effluent leaves the third chamber via an underdrain system located at the base of this chamber. The MCTT may be covered or uncovered, but if uncovered should be protected by a fence. The design of this device should be coordinated through the Headquarters Office of Storm Water Management – Design.

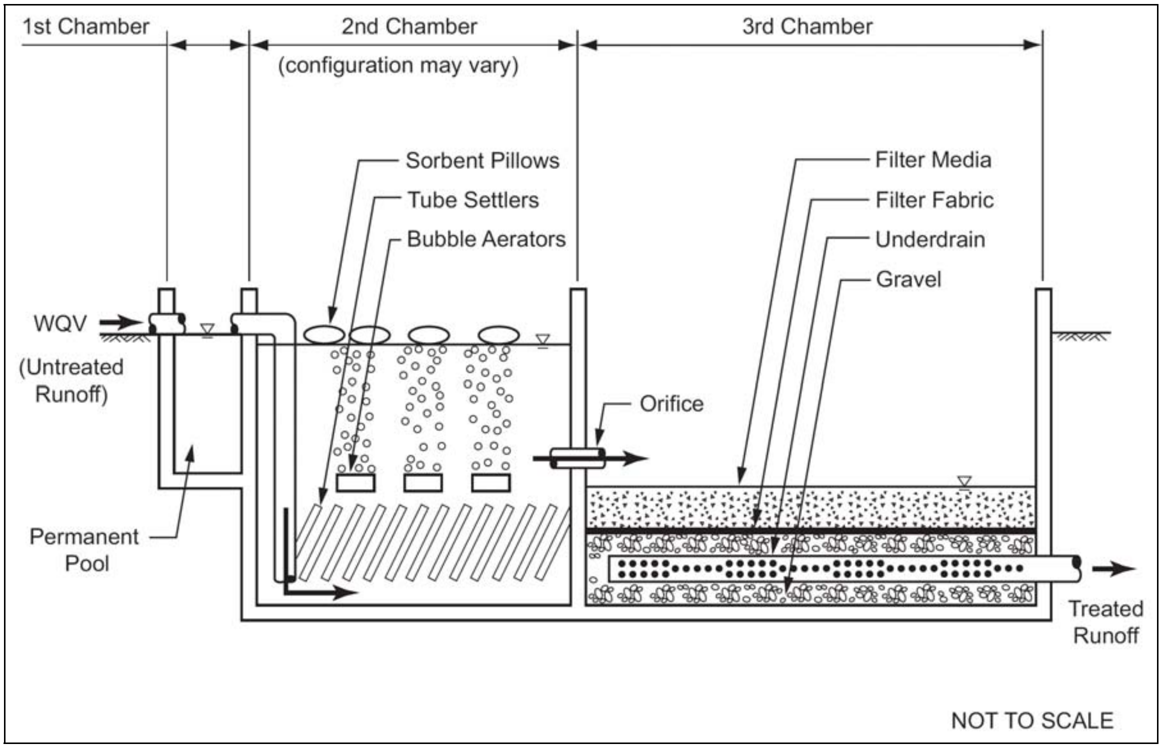

Figure B-17: Schematic cross section of an MCTT [17]

B.9.2 Appropriate Applications and Siting Criteria

The MCTT was developed for treatment of stormwater at critical source areas, such as vehicle service facilities, parking areas, paved storage areas, and fueling stations. To maintain longevity, potential sites should have a relatively high percentage of impervious surfaces contributing to the runoff, and runoff from the remaining area should not contain significant sediment. The WQV ≥ 123 m^3 (0.1 acre-foot [a-f]) for the MCTT to be considered.

Sites proposed for MCTTs must have sufficient hydraulic head to operate by gravity, approximately 1.5 m (5 ft), and are easier to place in flat to gently rolling terrain.

MCTTs should avoid locations having vector concerns because a permanent pool of water exists in the first chamber (and in the second chamber depending upon the outlet configuration); consult with local vector agency.

Upstream litter and sediment capture should be provided if possible, e.g., using biofiltration or a forebay.

[17] After CASQA Stormwater Best Management Practices Handbook - New Development and Redevelopment, January 2003.

B.9.3 Preliminary Design Factors

9.3.1 General Factors

Maintenance vehicle access to all chambers is required for inspection, periodic maintenance, and cleanout.

The maximum depth to invert of second chamber of 4 m (13 ft) below the ground surface, and Maintenance must be able to access invert at this depth.

Bypass overflow: off-line placement of Media Filters is preferred, but the first chamber should also have a separate overflow weir for events larger than the WQV, even if upstream diversion is provided.

The initial and second chambers should have a combined capacity ≥ 100% WQV, with the initial chamber usually having a minimum capacity of 25% WQV. The minimum volume of the third chamber is 75% of the WQV, with drainage time of 40 to 48 hours

Preliminary Design Factors for MCTT are summarized in Table B-9.

B.9.3.2 Sizing the MCTT Initial Chamber

The initial chamber should be sized to hold at least 25% of the WQV, with outflow using a weir, designed to pass a flow rate equal to the WQF (Water Quality Flow) appropriate for the location of the MCTT. Weir design should follow methods presented standard hydraulics texts. The depth of the initial chamber below this weir should be at least 0.30 m, to minimize resuspension of sediments.

A separate overflow weir also should be provided from the initial chamber, for storms larger than the WQV, using the same methods, and sized to pass the largest design event that will be passed through the device (up to the 100-year event). This weir generally will drain to the same downstream conveyance or water body as effluent from the third chamber of the MCTT.

B.9.3.3 Sizing the MCTT Second Chamber

The second chamber may be sized as small as 75% of the WQV, with outflow using weirs or orifices; however, the first and second chambers together should have a capacity ≥ 100% of the WQV. If outflow is made using an orifice, a minimum retention time of 24 hours should be employed, with orifice design following methods presented in Section B.4, Detention Basins, for the chamber volume and retention time selected. If the second chamber empties by weir, size the length of the weir to pass a flow rate equal to the WQF (Water Quality Flow) appropriate for the location of the MCTT. If site conditions allow, the second chamber may be sized to hold the entire WQV, at a retention time of up to 48 hours.

B.9.3.3 Sizing the MCTT Third Chamber

The third chamber is a filter bed; it should be sized to treat between 75% and 100% of the WQV. The size is first calculated for the area of the filter bed, then the length is determined, as the width is usually set by the width of the second chamber.

The equation for calculating the area of the filter bed (third chamber) is:

$$A_f = (FS \times C \times VOL \times d) / (k \times T \times [h + d]) \qquad \text{(Eq. 19)}$$

where

A_f	=	area of filter bed in the third chamber, m^2 or ft^3
FS	=	factor of safety, 2
C	=	conversion factor for units of permeability 100 for cm to m; 12 for inches to ft
VOL	=	75 to 100% of the Water Quality Volume, m^3 or f^{t3}
d	=	depth of filter bed, typically about 0.6 m[18]
k	=	coefficient of permeability of the filtering medium; metric: 10 cm/hr; US Customary units: inches/hr
T	=	design drain time for WQV, typically 40 to 48 hours
h	=	average water height above the surface of the media bed, taken as ½ the maximum head of the second chamber (distance to any overflow device from that chamber to the surface of the media bed); m or ft

The equation for calculating the length of the third chamber is:

$$L_{3rd\ chamber} = A_f / Width \qquad \text{(Eq. 20)}$$

where

$L_{3rd\ chamber}$ = length of the third chamber, m or ft

A_f	=	area of filter bed in the third chamber, m^2 or ft^2
Width	=	width of filter bed selected for design, m or ft

[18] Note that in the final design for the MCTT a gravel layer is placed below the sand layer. This layer has a thickness of 0.25 m, and it has within it the perforated underdrains. However, ignoring the correction for the effective vertical permeability of the stratified soil and gravel layers and the slight increase in depth of the combined layers introduces only minor error to the calculated area of the third chamber, given the large difference in permeability between sand and gravel, and can be ignored.

Table B-9: Summary of Multi-Chamber Treatment Train Siting and Design Criteria

Description	Applications/Siting	Preliminary Design Factors
Vault-type multi-chambered treatment device Treatment Mechanisms: • Sedimentation • Filtration • Adsorption and ion exchange (depending upon filtering media employed) Pollutants primarily removed: • Medium to fine sediments • Litter • Particulate metals • Other pollutants may be captured depending upon design of the second and third chambers	• WQV ≥ 123 m^3 (0.1 a-f) • Located at a areas as vehicle service facilities, parking areas, paved storage areas and fueling stations • Will perform better if the tributary area has a relatively high percentage of impervious area and/or a low sediment loading • Upstream litter and sediment capture should be provided if possible, e.g., using biofiltration or a forebay • Site must have sufficient hydraulic head to operate under gravity flow, minimum 1.5 m (5 ft) • Avoid locations having vector concerns as a permanent pool of water exists; consult with local vector agency. • More appropriate in flat to gently rolling terrain • Locate outside the 9 m (30 ft) Clear Recovery Zone, or consult with Traffic Operations to determine if guard railing is required	• Maintenance vehicle access to all chambers is required for inspection, periodic maintenance, and cleanout • Maximum depth to invert of second chamber of 4 m below ground surface; verify that Maintenance can access invert at this depth. • Bypass overflow: off line placement is preferred, but the first chamber should also have a separate overflow weir for events larger than the WQV, even if upstream diversion is provided; sized to pass the largest event that could be directed through the device (up to the 100-yr event). • The second chamber employs an outlet orifice or weir to pass the runoff to the third chamber • Minimum of 100% WQV combined capacity for the initial and second chambers • Minimum volume of the third chamber is 75% of the WQV, with drainage time of 40 to 48 hours • Third chamber filter media: 50% sand and 50% peat moss; for the sand: use Caltrans Standard Specification 90-3.03 for fine aggregate; Gravel: use Caltrans Standard Specification 68-1.025, Permeable Material, Class 1, Type B; Filter Fabric: Standard Specification Section 88-1.03, • Collector pipes: minimum 150 mm (6 inches) diameter laterals, and minimum 200 mm (8 inches) diameter collector pipe

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B.10 WET BASIN

B.10.1 Description

Wet basins are detention systems comprised of a permanent pool of water, a temporary storage volume above the permanent pool, and a shoreline zone planted with aquatic vegetation. Wet basins are placed in locations where naturally occurring wetlands do not exist. Wet basins are designed to remove pollutants from surface discharges by temporarily capturing and detaining the Water Quality Volume (WQV) in order to allow settling and biological uptake to occur. Wet basins are effective in removing sediments, nutrients, particulate metals, pathogens, litter, and BOD from storm water runoff. A schematic of a wet basin is shown in Figure B-18.

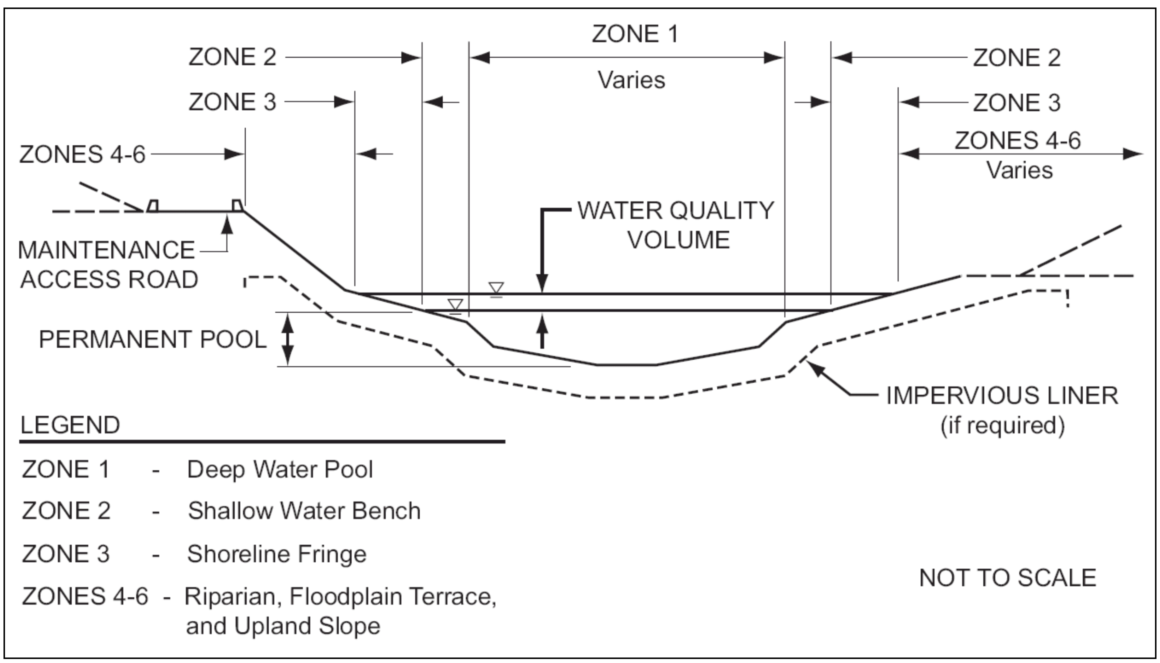

Figure B-18: Schematic of a Wet Basin

As indicated above, a wet basin has temporary storage capacity above the permanent pool for the Water Quality Volume. The WQV enters the wet basin and commingles with the permanent pool, during which time the water level in the basin rises to inundate the surrounding vegetation during a WQ event. The commingled water is slowly discharged through a water quality outlet device, usually an outlet riser, until the water level returns to the level of the permanent pool. To accommodate storms larger than the WQV design event, an upstream bypass or an emergency overflow outlet from the wet basin should be provided, sometimes from the same outlet riser. However, to provide additional safety in the event of failure of the upstream bypass or of the overflow outlet configured as a riser, an overflow spillway also should be considered.

The level of the permanent pool must be maintained year-round to support the plant community in the wet basin; this water level is maintained by connecting the wet basin to a stream channel, by seepage from springs, by placing the invert below the groundwater table[19], or by water from some other source. In arid climates, it can be difficult to maintain the proper level of the permanent pool using natural sources, and augmentation may be required. If 'gray water' is available nearby (gray water is water sold for non-potable use by a wastewater treatment facility, after receiving secondary or tertiary treatment), it could serve as a permanent source of water, but the use of potable water for the permanent pool is considered inappropriate in almost all situations due to its scarcity. As some infiltration might also occur, even for soils with a low infiltration rate, approval from the RWQCB must be obtained if gray water will be considered.

The depth of the permanent pool of water should have deep zones that prevent the growth of hydrophytic vegetation, and also to reduce the plan view of the basin.

Specific plant species suitable for inundated conditions are used in the shallow zones within the permanent pool, and these plants provide biological processes that aid in reducing the amount of soluble nutrients and for some dissolved metals. Other zones in the wet basin have vegetation more suited for the expected frequency of inundation (see the Hydrologic Conditions for Vegetation below).

Wet basins have the potential to attract and harbor sensitive or endangered species, which may prevent the maintenance activities needed to maintain the proper functioning of the basins and for vector control. Because of the potential for endangered/sensitive species establishment, the Department is required to contact the appropriate state and federal regulatory agencies early in the design phase to discuss the proposed location of every wet basin.

Design of a wet basin must be coordinated through the Headquarters Division of Environmental Analysis – Policy, Planning and Permitting, and Headquarters Design – Office of Storm Water Management.

B.10.2 Appropriate Applications and Siting Criteria

For wet basins to be considered, the design Water Quality Volume must exceed 123 m^3 (0.1 acre-foot [a-f]). The site under consideration for a wet basin should if possible be located where the visual aesthetics of the permanent pool is considered a benefit (such as a roadside rest area or vista point).

The proposed site must have a high water table or other source of water must be present to provide base flow sufficient to maintain a year-round plant community, even when considering losses due to infiltration and evapo-transpiration. The soil immediately below the invert must be relatively impermeable to limit loss of water by infiltration (NRCS Hydrologic Soils Group [HSG] soils C and D) unless a liner is used. Separation between seasonally high groundwater and basin invert should be > 3m (10 ft); use liner if separation between 0.3m and 3m (1 and 10 ft) unless approval by the RWQCB for placement without a liner is obtained.

[19] Approval from the RWQCB must be obtained for this placement.

The permanent pool volume should be at least 3x the Water Quality Volume, and additional temporary storage capacity greater than or equal to the Water Quality Volume, giving a minimum total volume of 4x WQV below the spillway elevation. Consult public health and vector control authorities; mosquito fish may be required in the wet basin. [20]

Conditions that do not allow for siting are: a site having hazardous soils or a contaminated groundwater plumes; objectionable backwater conditions in the storm drain system being induced; placement on or near unstable slopes, or slopes steeper than 15 per cent.

Note also that if the impounded volume exceeds 18 500 m^3 (15 a-f) the wet basin may classify as a jurisdictional dam, and be subject to other requirements; consult with District Hydraulics if the volume below the spillway exceeds this threshold.

The maximum width is suggested as 15 m, although if the width is greater than 7 m, access to both sides of the wet basin may be required; consult with the local vector agency regarding accessibility requirements around the wet basin.

Figure B-19: Wet Basin in District 11

B.10.3 Preliminary Design Factors

The wet basin must be employ an impermeable liner below the invert if placed in NRCS HSG A and B soils. Flows should enter the wet basin at low velocities, or use scour protection on inflow. Outfalls and spillways should also be provided with scour protection as necessary. Maintenance access around basin and paved or unpaved ramp to basin invert must be provided.

[20] The biological agent most commonly used to control mosquitoes is the mosquito fish, *Gambusia affinis*. Mosquito fish are most effective in wet basins that have a depth of 1.2 to 3.7 m (4 to 12 ft) and limited shallow shoreline (less than 30 percent of surface area); their effectiveness as a mosquito control agent declines greatly as the density of vegetation increases.

Upstream diversion channel or pipe for storms > WQV should be implemented if possible. The wet basin should have an upstream forebay to capture coarse sediment and litter if possible, and the site must allow ramps for maintenance access, with a volume of 10 to 25% of WQV.

Within the wet basin, a flow-path-to-width ratio of at least 2:1 configured in an irregular or meandering configuration must be provided. The invert of the wet basin may employ a 'micro topography' (contouring and benching of the invert to vary the water depth); care should be exercised to minimize stagnant areas (areas where incoming water does not displace or commingle with permanent pool). The basin may also be configured to fit the surrounding topography.

For the ground above the WQV elevation: use 1:4 side slope ratios or flatter for a minimum 3 m (16 ft) horizontally, with 1:3 side slopes maximum if approved by Maintenance. Below the WQV and the permanent pool elevation, the side slope ratios should be no steeper than 1V:3H, and 1:4 preferred along the entire the shoreline. Within the wet basin, average water depth should be approximately 1.2 to 2 m (3.9 to 6.6 ft), and typical maximum depth usually between 2.4 and 3.1 m (8 and 10 ft). Usually the shallow (vegetated) areas are limited to between 25 and 50% of surface water area of the wet basin.

The outlet used to discharge the WQV is designed to complete the drawdown in between 24 and 72 hrs, but typically 24 to 48 hrs. The WQ outlet should employ a debris screen (or equivalent) and riser similar to that shown in Figure B-7 on page B-29. In addition to a device that safely discharges the WQV, an outlet device must pass the largest event that could reach the basin, this may be done using the same device that will discharge the WQV, or by a separate device. Finally, some jurisdictions have more stringent requirements, and these should be consulted.

The wet basin should have a freeboard ≥ 300 mm (12 in), where freeboard is defined as the distance between the elevation at the top of the containment forming the basin, and the water surface elevation of the largest storm that can enter the basin; it is assumed that when that storm is passing through the wet basin, the initial water surface elevation in the wet basin includes the WQV retained above the permanent pool.

The WQ outlet using a perforated riser may be calculated using Equation 6, page B-30, taking the ($H-H_o$) term as equal to the height of the WQV above the permanent pool. The outlet for the largest storm that may reach the basin may be made using a weir or a pipe riser having a minimum nominal diameter of 900 mm (36 in.), or larger if District practice, designed using methods found in standard hydraulics references.

A drain for maintenance purposes should be placed if possible in wet basin, or defined sump area constructed for pumping during major maintenance.

The design for the wet basin must provide appropriate vegetation for each hydrologic zone. Native soils at invert may require added organics.

Consider fencing around the wet basin to restrict public access.

Preliminary design factors are shown in Table B-10.

Table B-10: Summary Of Wet Basin Siting and Design Criteria

Description	Applications/Siting	Preliminary Design Factors
• Impoundments where the WQV is temporarily detained in a permanent pool. Treatment Mechanisms: • Sedimentation/filtration • Adsorption to soil particles and by vegetation for certain contaminants Pollutants removed: • Total Suspended Solids • Nutrients* • Particulate Metals • Pathogens • Litter • BOD * Reductions observed for dry weather flow only. [End of this column]	• Minimum WQV > 123 m^3 (0.1 a-f) • Volume of water in permanent pool > 3x WQV • Should if possible be located where the visual aesthetics of the permanent pool is considered a benefit (such as a roadside rest area or vista point). • Permanent source of water must be available, and sufficient for all losses including infiltration and evapo-transpiration • Do not consider for sites with hazardous soils or contaminated groundwater plumes • Sufficient head to prevent objectionable backwater condition in the storm drain system • Preferred maximum width 15 m (49 ft) See footnote 24 • Consult public health and vector control authorities; mosquito fish may be required in the permanent pool of the wet basin • If the impounded volume exceeds 18 500 m^3 (15 a-f) consult with District Hydraulics to determine if the basin would classify as a jurisdictional dam • Not appropriate on or near unstable slopes, best sited in flat or gentle terrain of up to 15% slopes [This column continues on next page]	• NRCS HSG A and B soils at invert requires the use of an impermeable liner to maintain the permanent pool • Flows should enter at low velocities, or use scour protection on inflow; protect outfall and spillway with scour protection as necessary. • Maintenance access around basin and paved *or unpaved* ramp to basin invert [NB. Undergoing additional consideration.] • Upstream diversion channel or pipe for storms > WQV if possible; • Place if possible an upstream forebay for sediment and litter, with a volume of 10 to 25% WQV • Flow-path-to-width ratio of at least 2:1 configured in an irregular or meandering configuration • The invert may employ a 'micro topography' (contouring and benching of the invert to vary the water depth); care should be exercised to minimize stagnant areas (areas where incoming water does not displace or commingle with permanent pool) • Use 1:4 side slope ratios or flatter for area above the WQV for a minimum 3 m (16 ft) horizontally; 1:3 side slopes maxi. above this area if approved by Maintenance [This column continues on next page]

[21] If width is greater than 7 m, access to both sides of the wet basin may be required; consult with the local vector agency regarding accessibility requirements around the wet basin.

Table B-10: Summary Of Wet Basin Siting and Design Criteria (cont.)

Applications/Siting	Preliminary Design Factors
• Separation between seasonally high groundwater and basin invert > 3m (10 ft); use liner if separation between 0.3m and 3m (1 and 10 ft) unless RWQCB permission obtained. • Wet basins placed in cold climates will have reduced effectiveness • Locate outside the 9 m (30 ft) Clear Recovery Zone, or consult with Traffic Operations to determine if guard railing is required	• Internal (below the permanent pool) side slope ratio: no steeper than 1V:3H, and 1:4 preferred along the entire the shoreline. • Average water depth should be approximately 1.2 to 2 m (3.9 to 6.5 ft), and typical maximum depth usually between 2.4 and 3.1 m (8 and 10 ft). • Usually the shallow (vegetated) areas are limited to between 25 and 50% of surface water area. • Outlet design to drawdown the WQV within 24 to 72 hrs, typically 24 to 48 hrs • Downstream spillway or overflow riser: sized to pass the largest storm that can enter the basin (up to the 100-yr storm); minimum spillway length of 1 m, and/or minimum riser diameter of 900 mm (36 in.), or per District practice. Use local criteria for emergency flow passage if more stringent. • Provide freeboard ≥ 300 mm (12 in) (distance between the elevation of water in the basin when passing the largest storm that can enter the basin, and the elevation at the top of the confinement) • Discharge the WQV through an outlet riser and include a debris screen (or equivalent) • A 200 mm (8 inch) drain valve should be placed to evacuate water during major maintenance • Provide vegetation appropriate for each hydrologic zone in the wet basin • Native soils at invert may require added organics • Consider fencing around the wet basin to restrict public access

B.10.4 Hydrologic Conditions for Vegetation

Wet basins may have up to six specific hydrologic zones, as described in Table B-11. Local or native plant species should be used in all zones of the wet basin. Typically five to seven species of emergent wetland plants are used in the permanent pool. Large woody plants should not be allowed to be established in Zones 1, 2, or 3 of the wet basin. The District Office of Landscape Architecture should be consulted early in the design process to consider overall shape of the wet basin and plant materials for each hydrologic zone, if the design of the wet basin will be produced by the District. See also Caltrans *Technical Memorandum: Constructed Wetland Siting Study*, CTSW-TM-01-013, December 2001, page 5-10, for a list of native plants suitable for the shallow zones of wet basins (prepared for Caltrans Division of Environmental Analysis).

Table B-11: Wet Basin Hydrologic Zones

Zone number	Description and Topography	Hydrologic Condition and Water Depths Between Storm Events
1	Deep water pool (permanent pool; not used in all wet basins); volume of up to 25% of WQV; up to 35% of surface area (See Note 1); flat slopes, or slopes up to 1:3 where adjoining Zone 2	0.3 to 1.8 m; little or no plant growth in this zone, *especially between depths 0.5 to 1.0 m*
2	Shallow water bench (permanent pool); 35 to 75% of surface area; side slopes up to 1:3	0.15 to 0.3; hydrophytic plants in this zone
3	Shoreline fringe (could also include any upstream forebay to the wet basin); 25 to 40% of surface area; side slopes of up to 1:3	Regularly inundated during rainy season (conceptually, frequent storm events); this zone is sized to hold the WQV; depth depends is project specific; hydrophilic plants in this zone
4	Riparian fringe; side slopes of 1:4 (up to 1:3 if approved by Maintenance)	Periodically inundated (conceptually, up to 10 year storm events)
5	Floodplain Terrance; no set side slope ratio	Infrequently inundated (conceptually, > HDM design events)
6	Upland slopes; no set side slope ratio	Rarely or never

Note 1: Surface area is defined as the area at and below the elevation of Zone 3.

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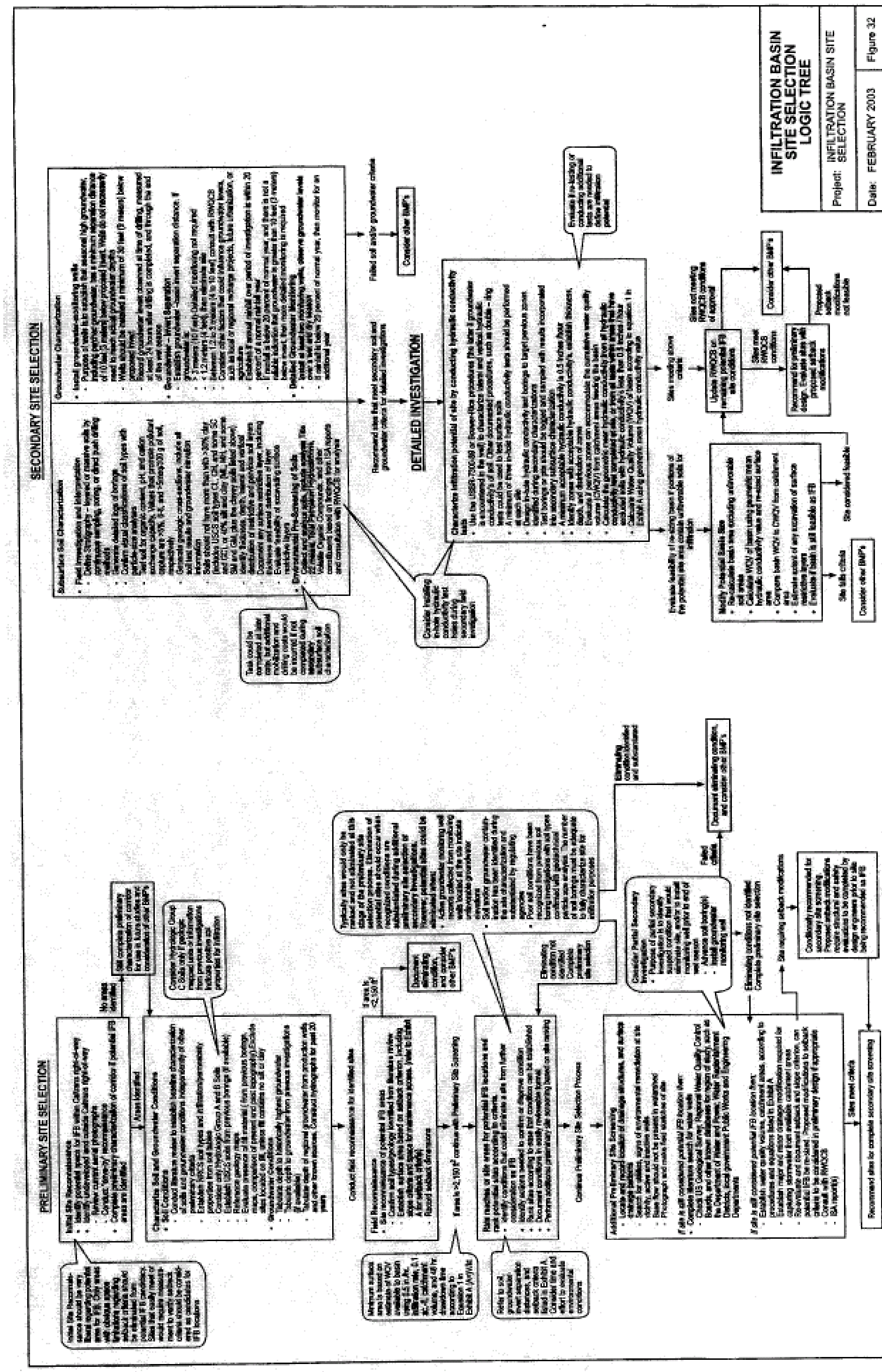
Figure B-20
See Footnote 22



22 Figure 20 from this handbook is taken from the "Infiltration Basin Siting Study, Vol. 1," CTSW-RT-03-025, Caltrans, June 2003, Figure 32. Available on-line:
http://www.dot.ca.gov/hq/env/stormwater/special/newsetup/_pdfs/new_technology/CTSW-RT-03-025/figures/FR_IFB_Figure_32.pdf

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